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Public Roads

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Volume 27

April 1952-February 1954

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

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Load cells under a free slab in the pavement, coupled with electronic equipment, measure axle weight, speed, and axle spacing of vehicles in motion

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

Weighing Vehicles in Motion

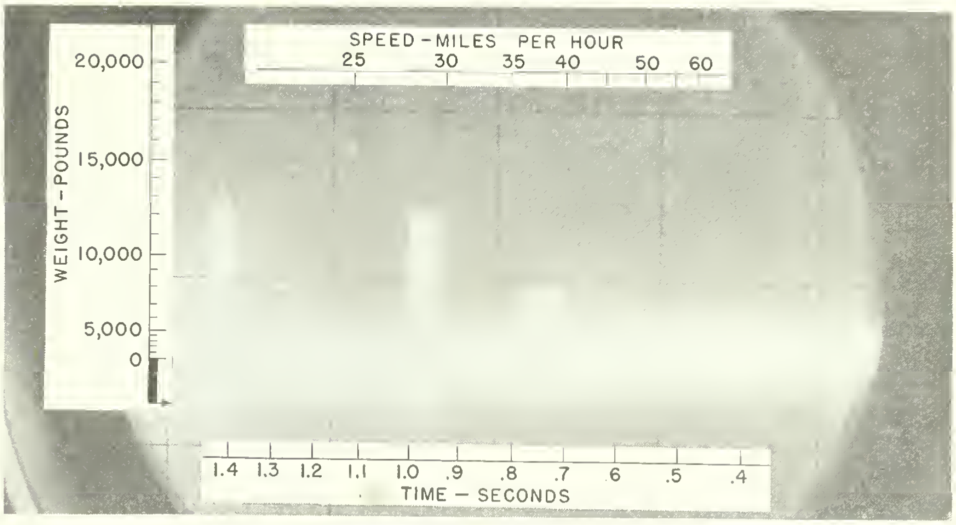


Figure 1.—Oscilloscope pattern of speed, axle-spacing, and axle-weight, with calibrated scales.

BY
THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by O. K. NORMANN, Chief, Traffic Operations Section, and R. C. Hopkins, Electrical Engineer

Comprehensive information on the gross weights, speeds, axle weights, and axle spacings of trucks traveling the Nation's highways is an important part of the vast body of knowledge necessary for the understanding and solution of our transportation problems. In the use of existing means of obtaining such data, involving portable loadometers and pit scales manned by relatively large crews, trucks must be stopped, and usually only a sample of the total traffic can be weighed and measured.

This article describes the development of an electronic weighing device which can measure axle weights, axle spacings, and speeds of vehicles moving at their normal operating speeds along the highway. The device consists of a narrow, free platform built in the surface of a traffic lane and supported by load cells containing columns to which are affixed groups of wire strain gages. The weight applied to the platform by a vehicle passing over it produces changes in the electrical currents flowing through the strain gages, and these changes, through electronic equipment, are reproduced as a pattern of light on an oscilloscope where they are photographed for record purposes.

A road detector tube, placed on the pavement at a predetermined distance in front of the platform, serves as a timing trigger for the sweep of the light spot across the oscilloscope, and from the recorded pattern not only axle weights but also axle spacings and the speed of the vehicle can be determined.

While the basic theory involved in the operation of this device is not unduly complicated, the experimentation and development work were beset with many electronic and structural problems. Many of these have been solved; others have been overcome in principle but are yet to be effected in practice.

For static weighing, the electronic scale is as accurate as the conventional lever-system pit scale. For highway-planning purposes, the electronic scale in its present stage of development is suitable for determining the frequency of different axle and truck weights, axle spacings, and speeds of vehicles without interfering with their normal operation. For enforcement purposes, through the use of an electronic scale of this type it is possible to cull out trucks with axle weights or total weights approaching or exceeding legal weight limitations.

There is good prospect that the electronic scale will be so improved that its accuracy in weighing vehicles in motion will approach still more closely the accuracy already attained in static weighing. It is also probable that simpler and less costly electronic equipment can be designed, and an oscillograph may prove an adequate substitute for the oscilloscope and recording camera.

KNOWLEDGE regarding the weights of trucks traveling over the Nation's highways is a prerequisite to a thorough understanding of our national transportation problems. Trucks are weighed to obtain information on the tonnage of freight carried by the several vehicle types on the various highway systems, to determine the magnitude and frequency of axle loads for highway design purposes, and to enforce license, tax, and load-limit laws.

At present, weighing trucks for highway planning and design purposes generally involves some sampling process. The most common sampling technique presently employed requires a six-man party equipped with portable loadometers. Such a party can seldom weigh more than 200 trucks per 8-hour day, at a cost of from $100 to $150. Truck operators object to the delay sometimes encountered at a weighing station and frequently use circuitous routes to avoid the station, even though the data obtained are used only for highway planning purposes and no law enforcement is connected therewith.

Common practice in enforcement of legal weight limits requires that, at any enforcement station, all trucks be stopped and weighed, excepting perhaps those obviously empty or of very light weight. Inconvenience and sometimes considerable delay to all operators result, even though only a small percentage are overloaded.

In realization of the widespread need for a scale which can weigh vehicles at their normal highway speeds, the Bureau of Public Roads during the past several years has investigated the possibility of adapting various types of mechanical and electrical devices to this problem. Experimental vehicle-weight recordings were made, using several electronic systems and various weight-sensitive electrical components such as variable resistors of carbon granules, variable capacitors, and strain gages. Of these, the highly perfected resistance-wire strain gage seemed most adaptable to vehicle-weighing requirements.

Electronic scales for weighing stationary objects or materials with an exceedingly high degree of accuracy have been in use for a number of years. Electronic scales are now being used to weigh freight cars while traveling at slow speeds, and to weigh materials in hoppers and ladles or materials picked up by cranes. At the time that the electronic scale described in this article was constructed, however, there was no record of a successful attempt to weigh motor vehicles while traveling at their normal highway speeds.

Basically, the electronic highway scale consists of a free platform built in the surface of a traffic lane and supported by load cells. A small voltage output of the load cells is amplified to operate a recorder which shows the weight of either a static or dynamic load. The load cells employed, and their associated electronic equipment, normally comprise an aircraft weighing kit used by commercial airlines, the Air Force, and the Navy to weigh their planes periodically and determine their centers of gravity. Usually, a plane is jacked up to flight position with a load cell between each of the three jacks and the jack points. Each cell must be capable of measuring within an accuracy of 10 pounds for any weight up to 50,000 pounds. The manufacturer of the kit, Cox and Stevens Aircraft Corporation, Mineola, N. Y., has cooperated in furnishing and adapting these components to the highway scale. Other instruments used for the scale are commercially available.

This report of progress to date discusses the circuitry and physical features of the scale. Also included is a discussion of its present accuracy, the improvements made during the investigation to attain this degree of accuracy, and desirable electronic and structural modifications that will further improve the scale and reduce the complexity of operation.

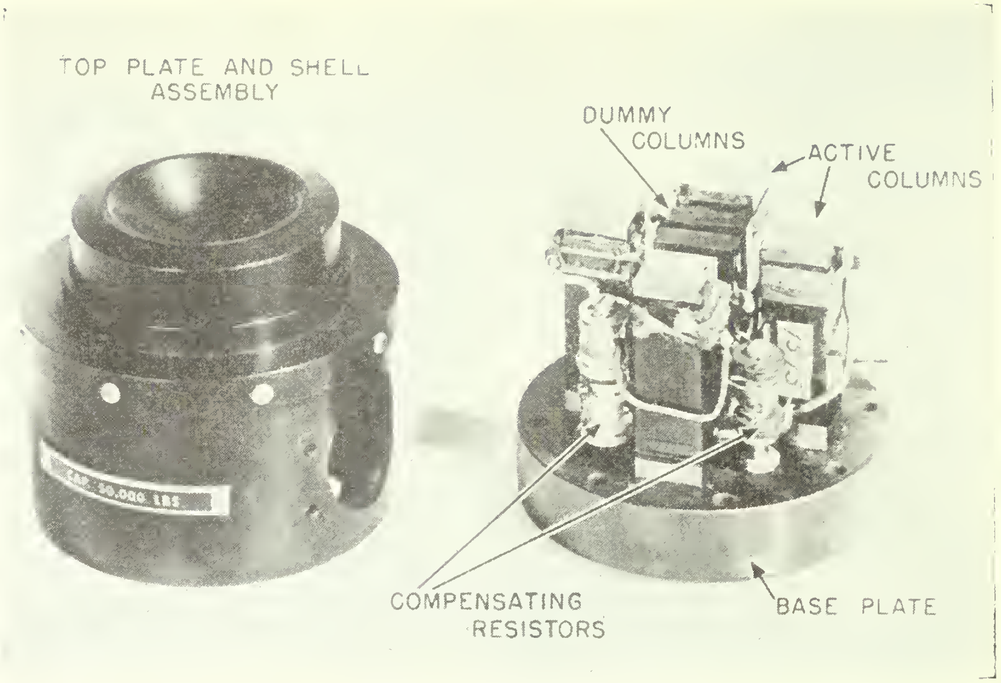


Figure 2.—The load cell.

Conclusions

The results of the tests made to date have been of sufficient scope to justify the following conclusions:

1. For static weighing, electronic scales in proper operating condition are as accurate as the conventional lever-system scales. Based on data obtained from these tests and other sources, electronic scales designed for the purpose for which they are used have a good record for retaining their calibration.

2. For highway-planning purposes, an electronic scale of the type described here is suitable for obtaining data on the frequency of different axle and truck loads, and the gross tonnage on a section of highway, without affecting the normal speeds of trucks. If desired, accurate measurements of vehicle speeds and axle spacings can be obtained at the same time.

3. For enforcement purposes, through the use of a scale of this type it is possible to cull out the trucks with axle loads or total loads which approach or exceed legal weight limits without affecting the normal speeds of most of the trucks. It is estimated, for example, that with a legal axle load of 18,000 pounds and a legal gross load of 50,000 pounds, all the trucks with illegal loads would be included in those which the scale recorded as having axle loads in excess of 16,000 pounds and gross loads in excess of 47,000 pounds. For enforcement purposes the trucks with loads approaching or in excess of the legal limits would have to be weighed statically to obtain a record which would be recognized in court at the present time.

4. There is a good possibility that the accuracy of the scale for weighing trucks in motion will be improved to approach still more closely the accuracy obtained by weighing trucks statically.

Electrical Principles

The electronic highway scale is a straightforward application of simple electrical principles. The experimental instrumentation is very intricate, however, and the circuitry of a practical scale may also be complex, although it need not be complicated in its operation.

The basic electrical principle upon which the electronic scale operates is that the resistance of a conductor is proportional to its length and inversely proportional to its cross-sectional area. The resistance-wire strain gage, usually consisting of several parallel wires connected in series and held solidly in alinement with a special cementing filler, makes use of this principle. When a strain gage is in turn bonded to some structural or mechanical part, a tension of the part in line with the strain gage will lengthen the wires and reduce their diameter, thereby increasing the resistance of the gage. Conversely, a compression of the part will reduce the gage resistance in proportion to the compressing force. The weight-sensitive load cell shown in figure 2 consists of columns to which strain gages have been bonded.

The electrical circuit of a load cell is a resistance network, commonly called a Wheatstone bridge, in which the current flow resulting from an applied voltage is divided equally so that there is no difference in potential at the output terminals. This resistance network consists of several wire strain gages mounted on columns so that compression of the columns deforms strain gages in opposite arms of the Wheatstone bridge, which in turn effects an unbalance in the resistance network, and consequently there is a difference in potential between the output terminals. The value of this potential difference is proportional to the compressing force.

Registering Components

To complete the electronic scale, a registering component must be provided and a means of calibrating the compressive forces

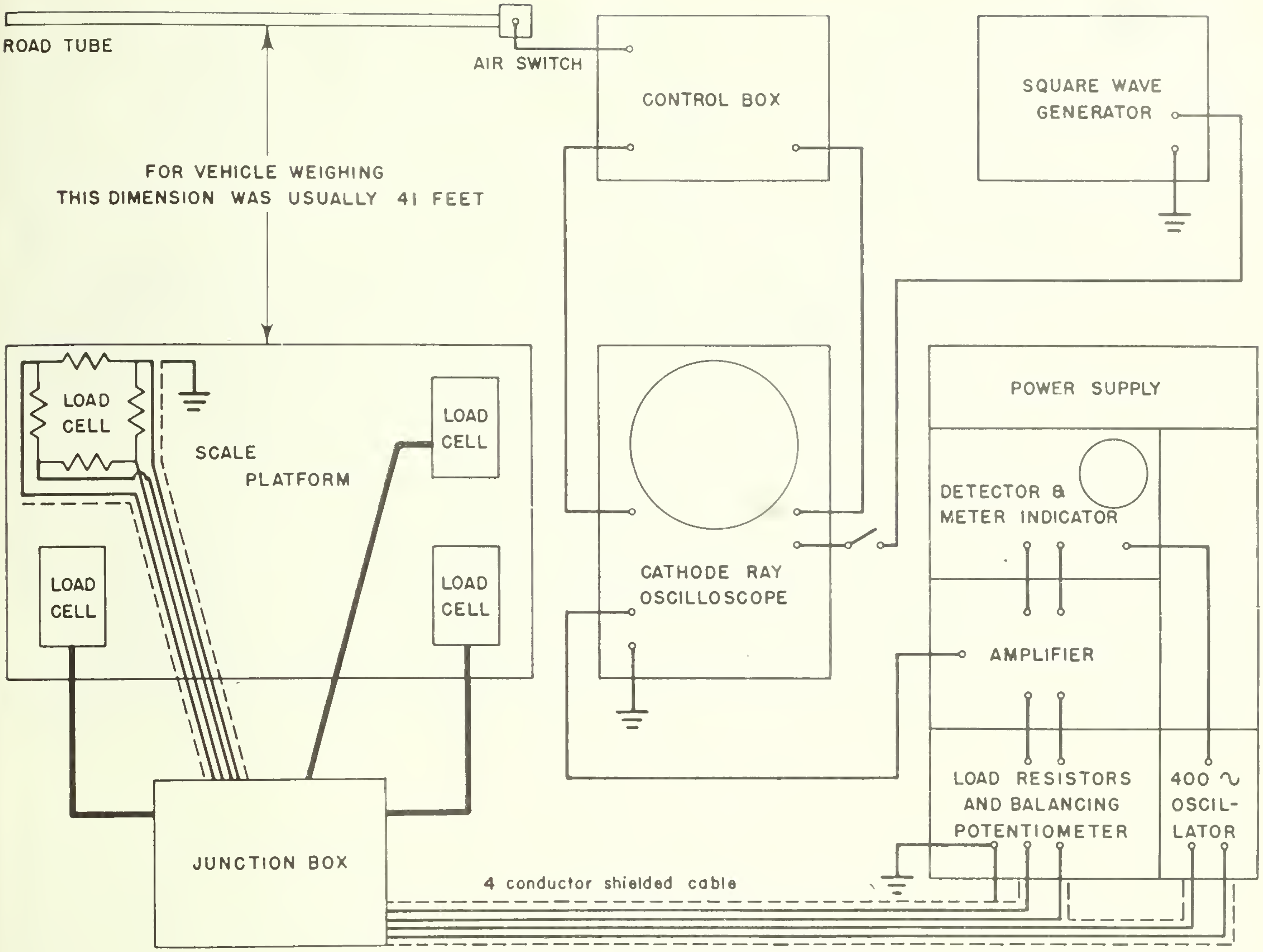


Figure 3.—Block diagram of the electronic scale.

is necessary. The cell output must also be sufficiently amplified to operate the recording device.

The short dummy columns of the load cell (fig. 2) carry no load. Strain gages mounted on these columns serve as the major temperature-compensating components in the cell and eliminate the need for adjustment of calibration with changes in temperature.

Four load cells were used in the experimental scale, as shown in figure 3. Each cell in itself was a complete Wheatstone bridge plus temperature and modulus-compensating resistors. The cells were connected in parallel so that the amplifier input would be a measure of the total unbalance of the four cells. This provided for the accurate indication of any weight, regardless of its position on the platform. A low-voltage 400-cycle current was supplied to the cell input because amplifiers for alternating current are generally more stable electrically than amplifiers for direct current. The cell output was fed through a range-control and calibrating-resistance network to a voltage amplifier and indicating meter. Also, a small voltage from the 400-cycle oscillator was fed directly to a phase-sensitive detector preceding the null-indicating meter so that the direction of the meter deflection showed whether the load was being increased or decreased. The range-control and calibrating-resistance network was composed of precision resistors so connected that known changes in cell unbalance could be directly produced to compensate for platform weight or other dead load on the scale and to calibrate the oscilloscopic presentation. The range-control and vernier potentiometer were also used to rebalance the bridge circuit for static weighing operations.

Weighing vehicles at normal highway speeds was accomplished by connecting an oscilloscope to the output of the second stage of the four-stage voltage amplifier included in the aircraft weighing kit. An oscilloscope contains a cathode-ray tube similar to a television picture tube. The cathode-ray tube in this case had a screen which would retain an image for a short time. A measure of the cells' unbalance was presented on the screen as a pattern which represented the weight for each axle of a vehicle. In addition, a photographic record was made of these transient phenomena.

The horizontal progression of the focused spot of light on the screen of the oscilloscope is called the sweep. The sweep speed is controlled by the rate of charge of a capacitor through a resistor, and only the flatter portions of this charging curve were usable as a time base. It was desirable, therefore, to provide a means for readily choosing a linear portion of the sweep. To accomplish this, a square-wave generator was used. This instrument produces an alternating current which changes from a constant positive value to a constant negative value in zero time as contrasted with the sine-curve wave form of conventional alternating current. The square-wave generator was connected to the Z axis of the

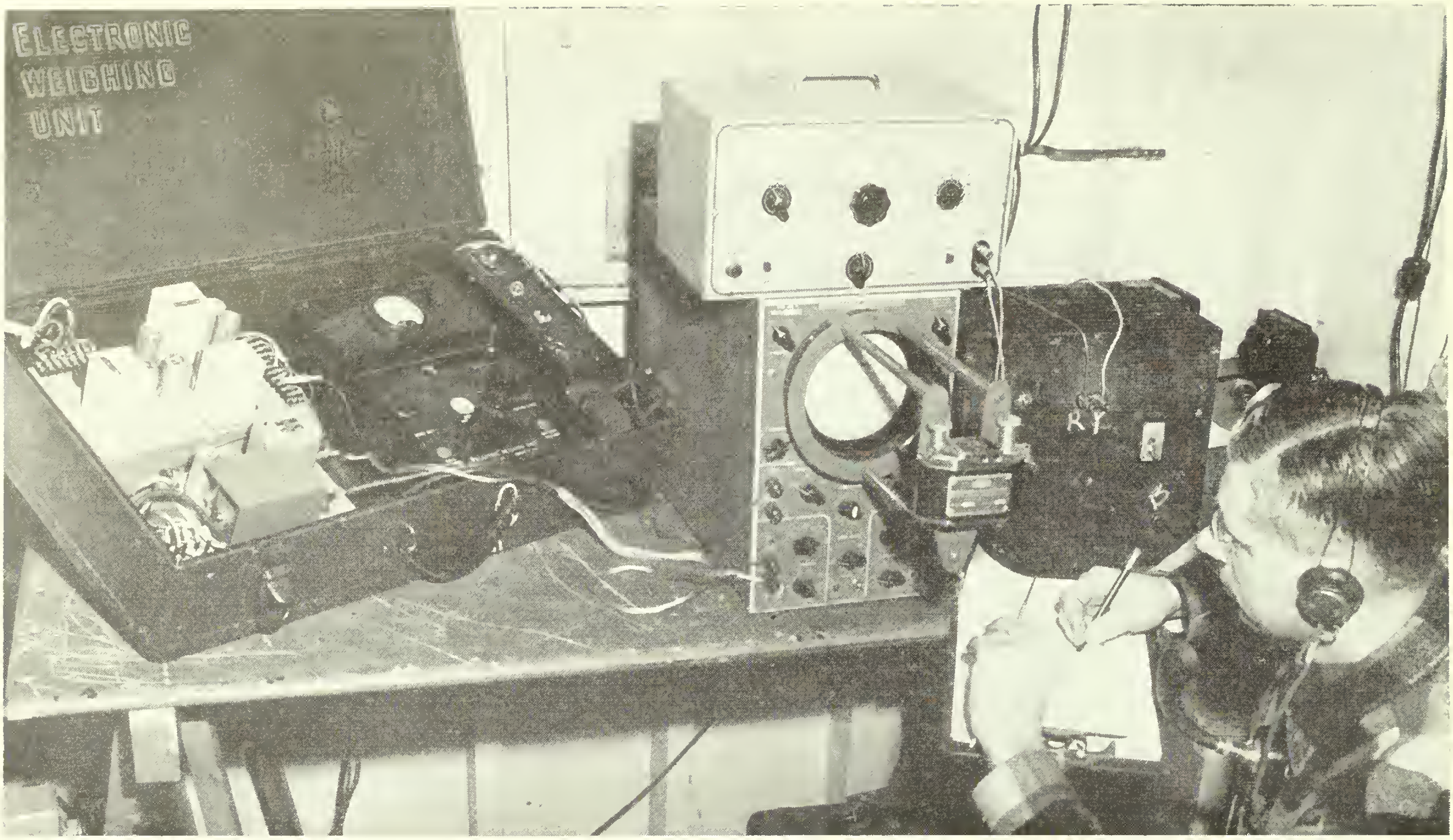


Figure 4.—The oscilloscope and associated electronic equipment in use.

oscilloscope so that chosen time intervals could be shown on the screen by blanking out the beam at a low-frequency rate. This produced a series of equal time dashes on the screen, and the sweep-generator controls could then be adjusted to provide linear time increments throughout the screen width. A rapid change could then be made from a slow sweep speed for recording all axles of a vehicle to a fast sweep speed for single-axle or tandem-axle recording and still provide records with abscissas directly referable to unit time.

To provide a tie between the sweep of the oscilloscope and the progress of a vehicle in the traffic lane, a road detector was used in front of the scale platform. The distance between the road detector tube and the platform was set so that the visible sweep of the oscilloscope coincided with the passage of the vehicle wheels across the scale platform. The front wheels of a vehicle passing over the road tube triggered the sweep circuit in the oscilloscope, permitting the focused spot as seen on the oscilloscope tube to sweep the screen at its predetermined rate.

As each wheel of the vehicle passed over the scale platform, the visible spot was also deflected vertically by a signal from the amplifier of the weighing kit. The amplitude of the vertical deflection was a measure of the axle weight. As the point of origin could be computed and rate of horizontal progression of the spot was known, the position of the weight indication of the front axle also was an indication of the vehicle speed, and the position of the weight indications of succeeding axles showed also a measure of axle spacing.

Operation of Electronic Scale

A typical set-up procedure will best describe the operation of the components of the weighing equipment. The weighing kit, oscilloscope, control box, and square-wave generator were placed conveniently on the bench, generally as shown in figure 4. The shielded cable from the load-cell junction box was connected to the weighing kit and the units were interconnected as shown in the block diagram (fig. 3). The road-detector tube was located at a distance in front of the scale platform, normally 41 feet, which was determined by the range in vehicle speeds. After connecting the air-switch cable to the control box, the set-up was complete.

It was found during the tests that the speed of the focused spot across the screen and the magnitude of its vertical deflection with a given load had a tendency to change during the first half hour of operation. To avoid errors due to changes in calibration because of instrument drift, a half-hour warm-up period was allowed prior to the start of each series of tests. During this warm-up period, the oscilloscope controls were adjusted within their operating range. The Y axis of the oscilloscope was calibrated in pounds by unbalancing the bridge circuit with the load switch in 5,000-pound steps. This method of electrically introducing an artificial load with the factory-calibrated weighing kit eliminated the need for calibrating the oscilloscope scale by placing actual known loads on the platform. The accuracy of this procedure was confirmed by field tests. Usually the Y-position, Y-attenuator, and Y-amplitude controls were set so that a 20,000-pound load caused a vertical deflection of 20 divisions on the screen. Deflection values for other weights were then recorded as shown by the typical weight-calibration record in figure 5.

A weight scale such as that appearing in figure 1 (p. 1) was computed from these weight calibrations. An axle weight is determined from it by alining the arrow at the bottom of the scale with the lowest point of the axle pattern and then reading the weight at the top of the pattern. The distance on the scale from the arrow to the zero reading represents the width of the pattern with no load on the scale.

The purpose of the study and the type of data to be recorded during each study determined the setting of the sweep speed. A two-second sweep time allowed all axles of trucks traveling within the normal range of speeds to be presented on the oscillo-

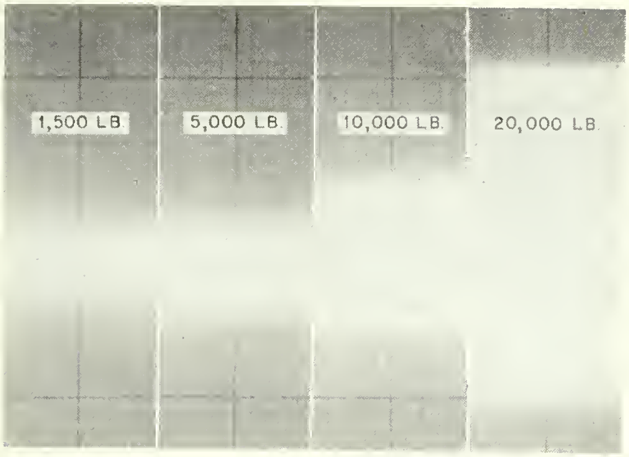


Figure 5.—Patterns of electrically introduced artificial loads for weight calibrations.

scope, whereas a one-fifth-second sweep time would permit a detailed presentation of single-axle or tandem-axle weights. An appropriate frequency was then selected on the square-wave generator: 20 cycles per second for the slower sweep and 50 cycles per second for the faster sweep. By observing the number of time marks per inch throughout the width of the oscilloscope screen, a combination adjustment of the sweep range, sweep vernier, X-position, and X-amplitude controls produced the exact linear speed desired.

Figure 6 shows a typical recording made before the final sweep adjustments had been made. Since the spot started its sweep at some point off the visible screen, a known time point was established by recording the axle weights of a passenger vehicle traveling the known distance from the road detector tube to the scale platform at a known speed. Knowing the road-tube spacing, vehicle speed, and the frequency of the equal time pulses, the time and speed scales which appear on figure 1 could be computed.

The weighing kit was provided with a balancing potentiometer which was adjust during the warm-up period to bring the null-indicating meter to center scale when the scale platform was unloaded. At the end of the warm-up period, a slight readjustment of the sweep vernier and the Y-amplitude control prepared the instruments for operation.

Any vehicle for which axle weights were desired could be selected by closing the switch on the control box and opening the camera shutter as the vehicle approached the road tube. After the passage of the vehicle the camera shutter was closed, the film was advanced, and the control-box switch was opened to prepare the instrument for the next vehicle to be recorded. With manual operation of the camera, the complete recording cycle required about 10 seconds. In occasional cases where two trucks were closely spaced, it became necessary to select the one for which data were desired.

A recording of axle weights and spacings and vehicle speed is shown in figure 1. The derivation of the time, speed, and weight scales superimposed on this figure has been previously described. Using these scales, it can be determined from the pattern made by the vehicle that it was traveling 39.5 miles per hour, had axle weights of 7,500 pounds, 12,200 pounds, and 13,300 pounds, and had axle spacings of 13.5 feet and 25.6 feet. The axle spacings were calculated after knowing the time spacings between the axles and their speed.

For static weighing, the road tube, control box, oscilloscope and camera, and square-wave generator were not used. The weighing kit was checked for balance and adjusted by the balancing potentiometer. One axle of the vehicle was run onto the platform, other wheels were trigged, the brakes released, and the null-indicating meter was brought back to center scale by adjusting the weight-indicating dials which were calibrated in pounds. The weight shown on these direct-reading dials was then the weight of the axle on the platform.

This experimental scale was constructed entirely with commercially available instruments to test an electronic system for dynamic weighing. No major electrical changes were made in the individual units. The experience gained from the experimental scale now makes it possible to draw general electronic specifications for a scale designed for dynamic weighing which would provide reliability, accuracy, and simplicity of operation.

Recommended Improvements

Some of the more important features that should be included in an improved instrument designed for weighing vehicles in motion which are relatively simple to accomplish are:

1. Redesign of the control box to provide for automatic operation. To accomplish this would require only the addition of one circuit which would reset the sweep trigger when the spot had completed its traverse of the oscilloscope screen.

2. Design of cathode-ray tube circuits specifically for the scale. The oscilloscope now in use is a general-purpose instrument and therefore contains many features not used in conjunction with the highway scale. Also, the present instrument has a greater range of control over the circuits that are used than is desirable. The controls should be reduced to those for spot position, spot intensity, focus, weight calibration, and sweep calibration. The last two mentioned should each have only a single control dial. These should be calibration adjustments rather than complete controls of the amplifier gain and sweep speed now available in the oscilloscope.

3. Automatic operation of the camera, and a large film magazine.

4. Redesign of the weighing-kit circuits to provide a well-shielded oscillator, a stable amplifier with a simple gain calibration and adjustment, and a vacuum-tube de-

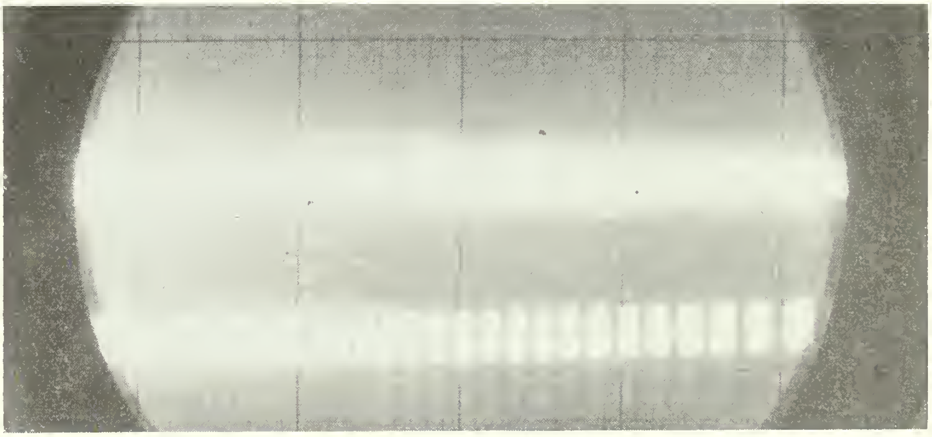


Figure 6.—Pattern produced by an automobile moving 40 miles per hour with 20 time marks per second inserted for speed and axle-spacing calibration.

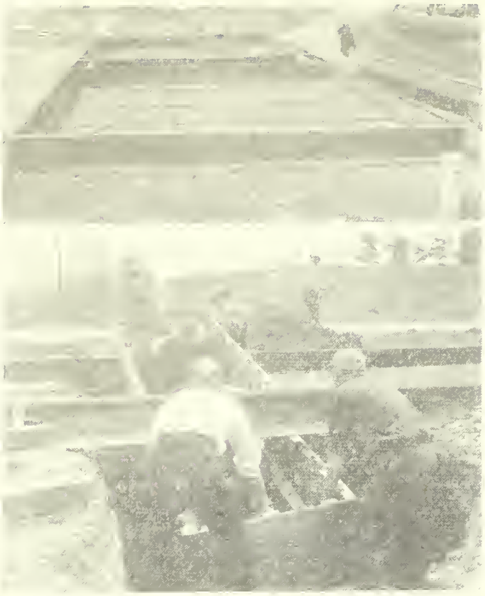


Figure 7.—Above: Steel framework used to form the pavement slab in which the weighing pit was installed. Below: Only an inside form was used for the pit.

tector so that the oscilloscopic presentation of weight is in one direction only from the zero, making for a single scale reading for each axle weight.

5. Possible substitution of a pen or heated-stylus recorder for the cathode-ray oscilloscope and camera.

Several of these design changes are now being incorporated in the present scale. Structural improvements are discussed at the end of the article.

Construction of the Scale

Concurrent with the electronic development, and of equal importance, a program of research was being conducted to determine the most suitable structural and mechanical features of the complete scale.

In April 1951, when it appeared from a number of rather crude preliminary tests that there was a possibility of weighing vehicles in motion by an application of electronic load cells, the last section of concrete pavement on the Shirley Highway, a freeway in Virginia just south of Washington, D. C., was nearing completion. This offered an opportunity to construct an electronic scale for a minimum cost at a location that was ideal for development and test purposes.

With the cooperation of the Virginia State Department of Highways and the contractor, the Williams Construction Company of Norfolk, Va., the Bureau of Public Roads completed the construction of the pit and slab necessary for the installation of the load cells and electrical equipment on May 7, 1951. The exact location of the installation is on the Shirley Highway 1,000 feet north of the intersection with U S 1 near Woodbridge, Va.

The design of the experimental scale was influenced by several considerations which now appear to be either elementary or of slight importance. Since the possibility of recording accurate weights of vehicles in motion seemed rather remote, due to the spring action of the vehicles and impact factors involved, it was obvious that an elaborate installation was not justified. It was necessary, however, that the installation be constructed in such a manner that it would be adequate for an accurate test of the equipment and principles involved. It was also essential that the original installation be designed so that changes could conveniently be made in the electrical equipment and in the suspension of the floating slab, as the investigation progressed, without the need of major changes or reconstruction of the pit located in the roadway. Furthermore, it was desirable that the tests be conducted without unduly interfering with normal traffic operations on the freeway.

The floating slab supported by the load cells, and the pit under the floating slab which provides access to the load cells, were constructed in the right-hand lane of the two lanes used by southbound traffic. The highway carries an average of about 50 dual-tired trucks per hour in each direction, with a fairly uniform flow day and night. Most of the trucks normally use the right-hand lane at the location and are weighed by the Virginia Department of Highways on their conventional lever-system scales located 4 miles to the south. As a check on the accuracy of the electronic

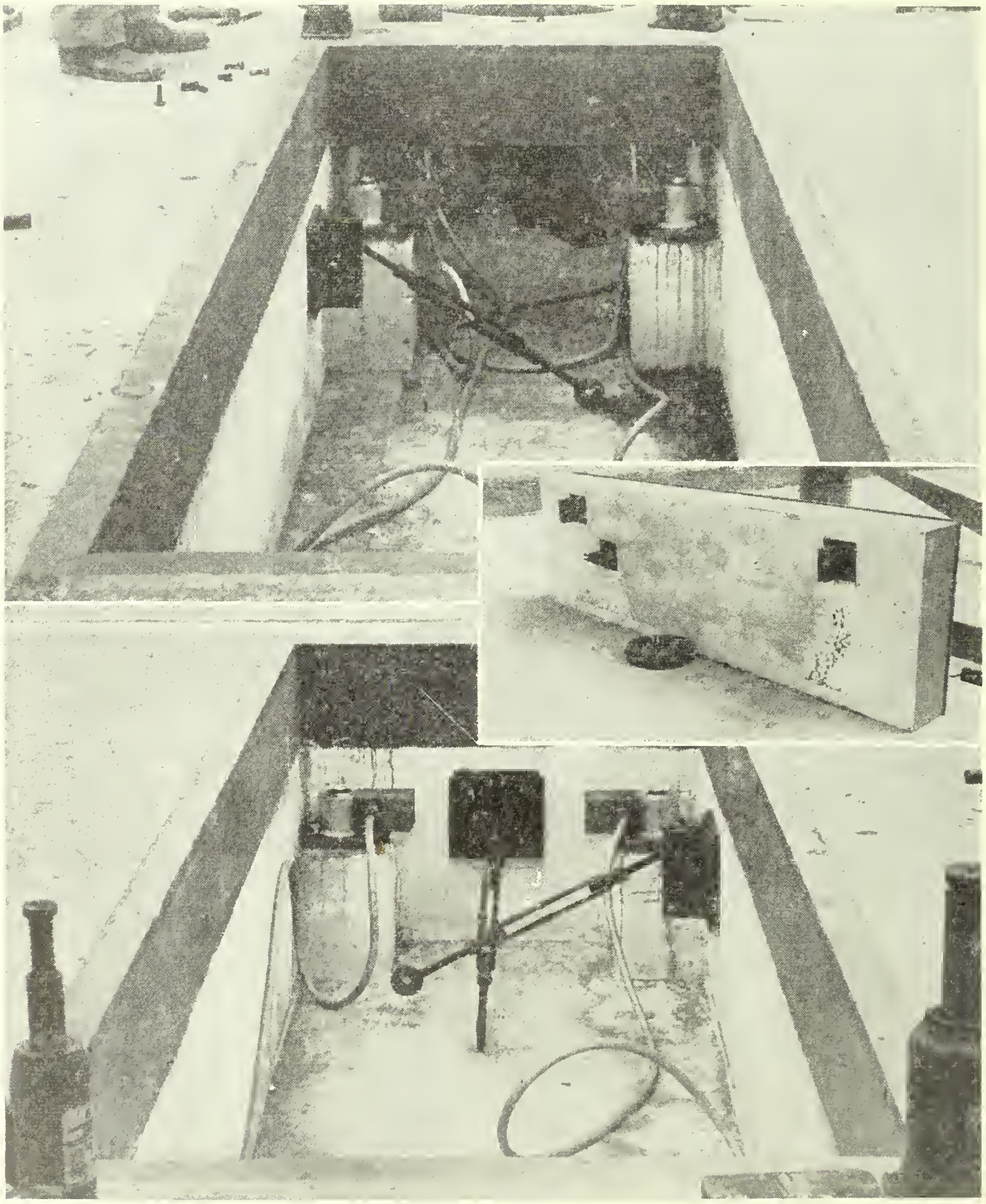


Figure 8.—The completed pit, showing the load cells in place and the turnbuckles attached to the pit brackets. The west end and the access manhole appear in the upper view; the east end is shown below. The floating slab, with the turnbuckle brackets attached to the bottom face, is shown in the inset.

scales for weighing vehicles in motion, the static weights of the trucks were obtained at the lever-system scales.

Maximum possible smoothness of the pavement at the electronic scale was obtained by finishing the surface to the top edge of two 30-foot steel channels rather than to the top of conventional 10-foot concrete forms. A framework of 10-inch channels, as shown in figure 7, was constructed to form the outside dimensions of one concrete slab 30 feet long and 12 feet wide, and the top of the inside walls of the pit. This framework was placed inside the conventional concrete forms and remained in place after the conventional forms were removed. One-inch steel bars spaced about 3 feet apart in both directions were welded to the channel framework to supplement the steel-mesh reinforcing used throughout the project and to assure continued smoothness of the surface at the scale. Extra care was also taken to obtain a smooth surface on the two 30-foot slabs adjacent to the one in which the scale was located.

The pit to accommodate the floating slab and load cells occupied the central 10 feet of the normal 12-foot traffic lane and was 37½ inches wide to allow a three-fourths-inch clearance between the floating slab and the walls of the pit. It would have been desirable to construct the pit for the entire 12-foot lane width, but this would have introduced additional construction difficulties that did not seem warranted for this investigation. The 36-inch width for the floating slab was selected to obtain the maximum size on which the weight of not more than one axle of a vehicle would be present at any one time. The clear distance between tire-contact areas on dual axles approaches a minimum of 3 feet. The thickness of the floating slab was 12 inches. It was reinforced with 12-inch steel channels weighing 35 pounds per foot which had been welded together to form the outside dimensions. The ends of 1-inch reinforcing rods spaced about 2 feet apart were also welded to the channels in addition to the two layers of wire mesh.

The depth of the pit was 30 inches. This dimension was limited by the elevation of the bottom of the drainage ditch of the roadway. A greater depth for the pit would have been desirable if natural drainage from the bottom of the pit could have been provided. The bottom and walls of the pit were 12 inches thick. They were poured integrally with the surface of the rest of the 30-foot slab. Access to the pit with the floating slab in place was through a manhole located on the shoulder, as shown in the cover illustration.

Two steel angle brackets were fastened on the long side of the pit and one on the short side, and corresponding brackets were bolted to the under side of the floating slab. These brackets were connected with tie bars and turnbuckles, to hold the slab in proper horizontal position. After the slab was installed a ¾- by 1-inch rubber strip was placed in the space between the slab and the walls of the pit.

The location of the four load cells and the turnbuckles for adjusting the tie bars that held the floating slab in its proper horizontal position is shown in figure 8. The cover illustration shows a view of the completed installation as it appeared on the surface of the road.

Initial Tests of Accuracy

A loaded two-axle dump truck was used in the first series of tests to determine the accuracy of the scale. The static weight of

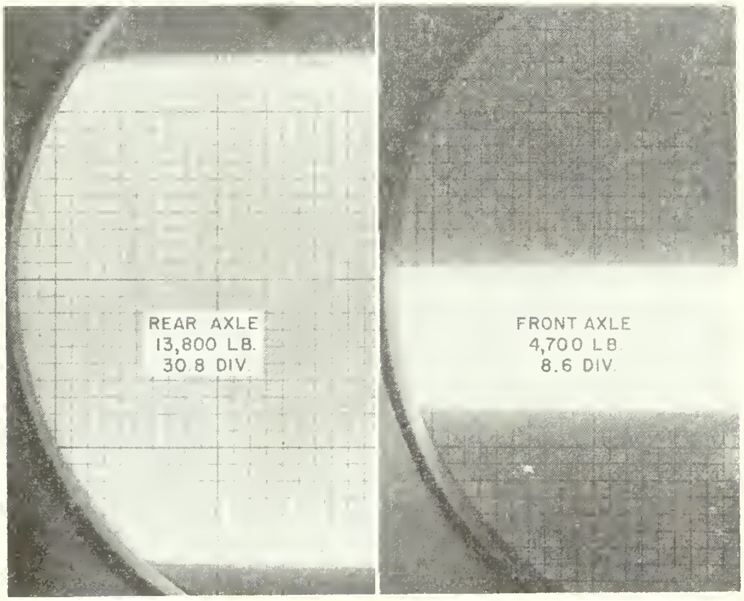


Figure 9.—Static weight pattern for a two-axle truck.

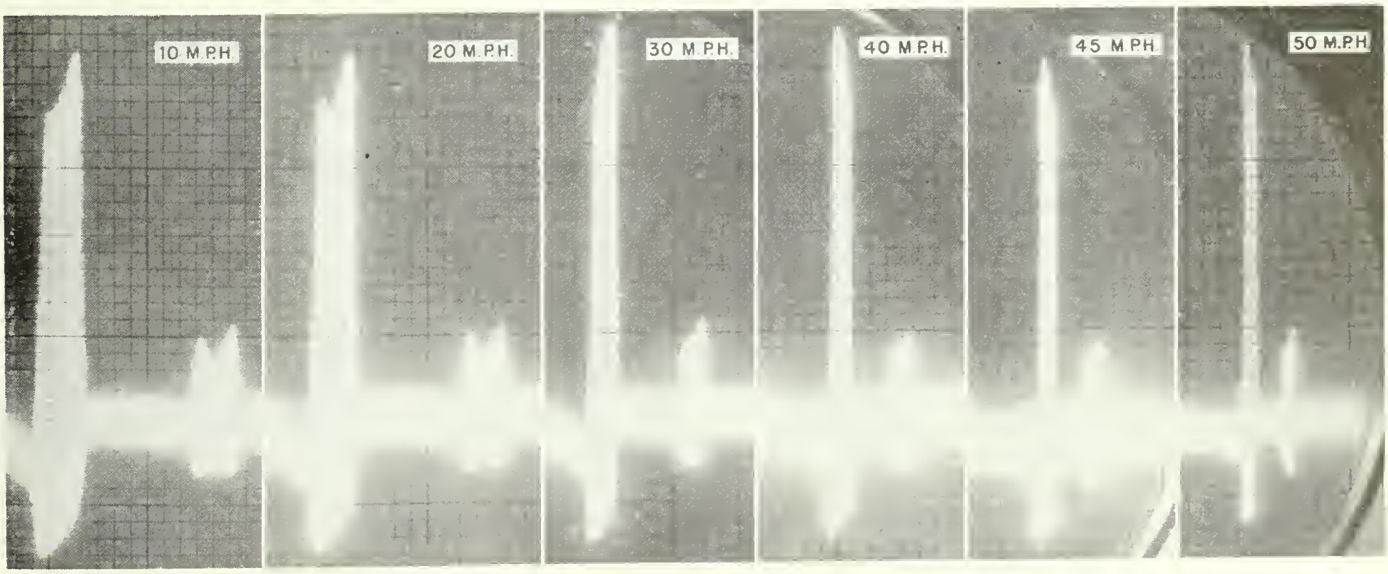


Figure 10.—Weight patterns for various speeds of the truck weighed statically in figure 9.

Table 1.—*Recorded weights of two-axle truck at various speeds*

Speed	Based on longest vertical oscillation				Based on extremes of pattern			
	Front axle		Rear axle		Front axle		Rear axle	
	Recorded weight	Error [1]	Recorded weight	Error [1]	Recorded weight	Error [1]	Recorded weight	Error [1]
M.p.h.	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*
0	4,700		13,800	....	4,700	......	13,800	......
10	4,800	+100	12,800	−1,000	4,950	+250	13,850	+50
20	4,600	−100	12,700	−1,100	4,750	+50	13,500	−300
30	4,700	0	13,400	−400	4,750	+50	13,900	+100
40	4,300	−400	13,700	−100	4,300	−400	13,800	0
45	4,750	+50	13,300	−500	4,750	+50	13,300	−500
50	4,900	+200	13,200	−600	4,900	+200	13,200	−600

[1] Deviation from static weight.

each axle of the truck was obtained first on the electronic scale and then on the conventional lever-system scale. The oscilloscope pattern of the static weight for each axle while the truck was standing on the electronic scale is shown in figure 9. It is possible to vary the magnitude of the vertical deflection of the oscilloscope spot for a given load by turning the Y-amplitude dial. For example, it is possible to change the adjustment so that one division of the grid on the screen will represent 10 pounds, 10,000 pounds, or any load between 10 and 10,000 pounds. During this series of tests the Y-amplitude was adjusted so that the pattern for the load of the heavier axle of the truck covered most of the screen. This permitted the recorded weights as obtained from the patterns to be read with a greater precision than if a smaller scale were used to represent the same weight.

After obtaining the static-load pattern, a series of weight patterns was obtained as the truck crossed the electronic scale at different speeds, ranging from 10 to 50 miles per hour. These patterns are shown in figure 10. The difference between the magnitude of the static-load pattern for each axle, measured vertically, and the magnitude of each pattern while the truck was in motion is a direct measure of the error in the recorded axle weights. Table 1 shows the recorded weights of the truck traveling at various speeds as obtained by translating the oscilloscope patterns into axle loads.

There was some question during these initial tests as to whether the recorded weights were represented by the magnitude of the extremes of the axle-weight patterns or by the magnitude of the longest vertical lines of the patterns. One set of figures in table 1 is based on the longest vertical lines within the patterns, whereas the other set of figures is based on the extreme magnitudes of the patterns. In this test, the weights that are based on the extremes of the patterns correspond more closely with the static weights than those that are based on the longest vertical lines. This did not hold true in subsequent tests. Generally, the weight represented by the longest vertical line of a pattern corresponds more closely with the static weight than the weight represented by the extremes of the pattern. In either case, the maximum difference between the static weight and the recorded weight while the vehicle was in motion was 400 pounds for the front axle, a difference of 8.5 percent. For the rear axle, the maximum error was 1,100 pounds or 8.7 per cent in the one case and 600 pounds or 4.4 percent in the other case.

Most of the errors shown in table 1 are within the precision with which it is possible to read the magnitude of the oscilloscope patterns.

Major Errors in Dual-Axle Weights

It should be noted that there is no consistent increase or decrease in error with an increase in the speed of the vehicle. A number of tests of this type were made, using two-axle trucks, during the period that improvements and adjustments were being made in the electrical equipment and in the procedures being developed for conducting more comprehensive studies of normal traffic using the highway. Similar results were obtained in all of these tests, there being no indication that a greater error was likely to occur at one particular speed than at another speed. Many minor improvements were made in the equipment during this period, but attempts to increase the accuracy by minor changes in the electrical apparatus were not successful. The

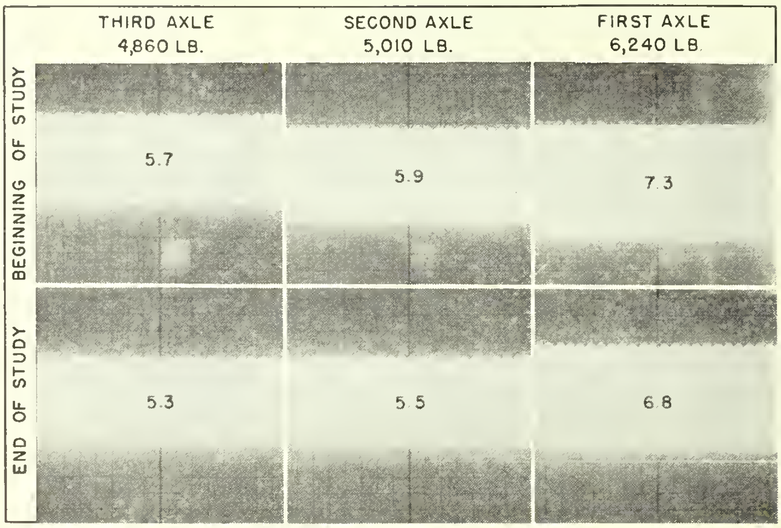


Figure 11.—Static weight patterns for a three-axle truck at the beginning and end of the study. The figures in the pattern bands indicate their widths in divisions.

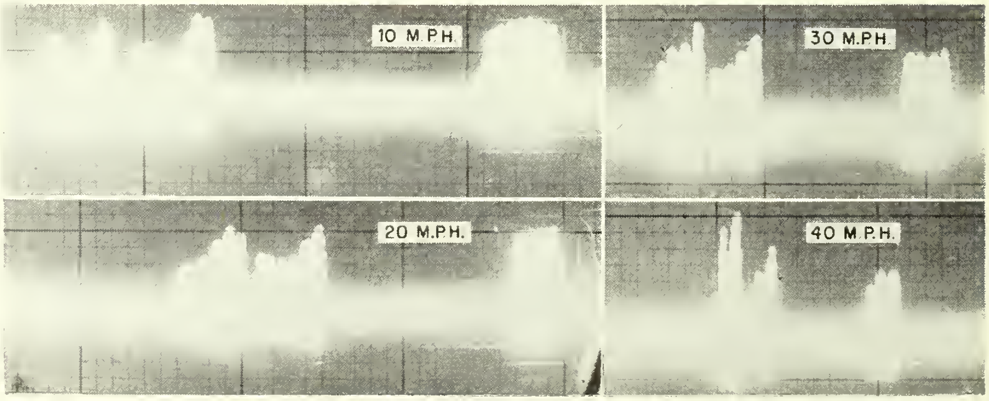


Figure 12.—Weight patterns (with unequal dead weights on load cells) for various speeds of the truck weighed statically in figure 11.

use of equipment built especially for weighing vehicles in motion rather than the present equipment, which was a combination of instruments already available commercially, appeared to be the only way in which a greater accuracy could be attained. The results obtained, however, appeared to be sufficiently accurate to justify a more extensive test on the large variety of truck types using the highway.

Table 2.—*Recorded weights of three-axle truck at various speeds*

Speed	First axle		Second axle		Third axle	
	Recorded weight	Error [1]	Recorded weight	Error [1]	Recorded weight	Error [1]
M.p.h.	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*
0	6,240		5,010		4,860	
10	6,500	+260	6,700	+1,690	6,500	+1,640
20	6,450	+210	6,900	+1,890	6,800	+1,940
30	6,250	+10	7,350	+2,340	8,550	+3,690
40	6,150	−90	7,700	+2,740	9,850	+4,990

[1] Deviation from static weight.

The results of the initial tests, in which trucks using the highway were weighed by the electronic scale as they passed the study location and were again weighed statically on the conventional lever-system scale, showed that extremely high errors were obtained for trucks with dual axles. To determine whether speed had any effect on the magnitude of the error in the weight of dual axles, a series of tests similar to the initial tests on two-axle trucks was conducted on a three-axle single-unit truck. The patterns obtained for the three-axle truck when it was operated at various speeds are shown in figures 11 and 12.

Figure 11 shows the static weight patterns for each of the three axles at the beginning of the study and also at the end of the study. The difference between the magnitudes of the patterns at the beginning of the study and at the end of the study illustrates the drift of the calibration when tests are started without allowing a half-hour warm-up period.

Figure 12 shows the patterns obtained as the three-axle truck crossed the scale at 10, 20, 30, and 40 miles per hour. Translating these patterns into weight measurements resulted in the data shown in table 2. It may be seen that the speed of the truck had no consistent effect on the accuracy of the recorded weights for the first axle, the errors in each case being within the precision with which the magnitude of the recorded patterns can be measured. The results for the second axle (table 2) show a marked increase in the magnitude of the error with an increase in speed of the truck. This is also true for the errors in the recorded weight of the third axle. The errors for the third axle at the higher speeds are considerably greater than the errors for the second axle.

Improvements in Slab Suspension

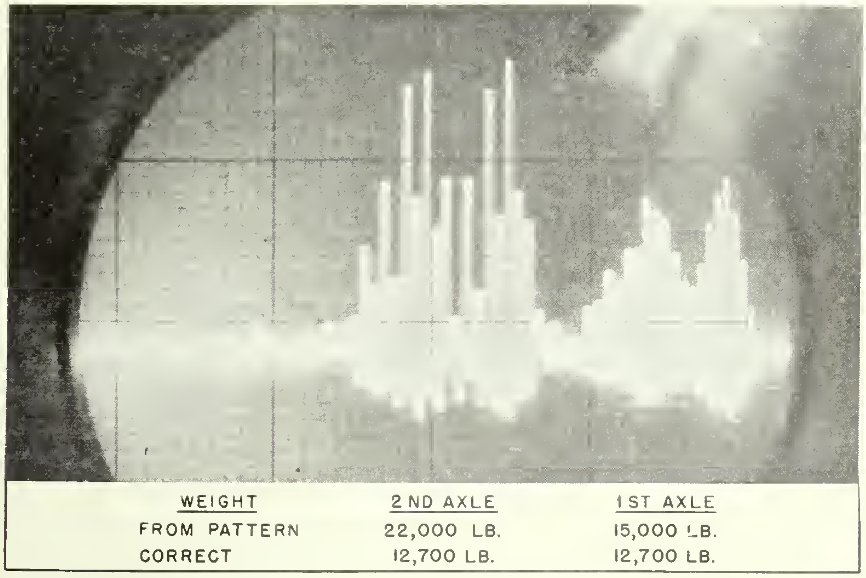
Some horizontal and vertical movement of the floating slab as the truck passed over it was noticed during the period in which the series of tests on the three-axle truck was conducted. The movement of the slab appeared to be causing impacts which were transmitted to the load cells. The impacts were definitely greater as the second and third axles went over the scale than with the passage of the first axle of a truck, especially at high truck speeds.

It may be seen from figure 12 that the appearance of the patterns for the second and third axles of the truck is quite different from the appearance of the patterns for the first axle, especially at the high speeds. It seemed desirable, therefore, to obtain more detailed pictures of the weight patterns. Figure 13 is such a picture of a pattern for the dual axle of a semitrailer traveling 53 miles per hour. It was obtained by increasing the horizontal speed of the oscilloscope spot as it swept across the screen. In this case, the spot moved across the screen in one-fifth of a second and the dual axle was on the 3-foot scale platform for only one-tenth of a second. To obtain such a weight pattern approximately centered on the oscilloscope screen requires split-second timing. The oscilloscope spot, which starts its sweep off the screen, must be triggered so that the one-tenth-second weight pattern falls within the one-fifth-second visible sweep time.

Each vertical deflection of the oscilloscope spot may be seen as a separate line in figure 13. The spot traveled up and down at the rate of 400 oscillations per second, the magnitude of its vertical travel being controlled by the weight on the load cells. It therefore appeared, from a study of figure 13, that the load cells were being subjected to extremely rapid changes in load, possibly caused either by a horizontal movement of the slab, by a rocking of the slab on its four supports, or by vibrations in the slab, produced by the first axle of the tandem, that were still remaining at the time the second axle passed.

Complete elimination of any vibratory movement of the floating slab, both vertically and horizontally, seemed desirable. The horizontal movement was eliminated by welding the turnbuckle tie bars to the brackets on the under side of the slab and on the walls of the pit. Previously these tie bars had been pin-connected and the pins had developed some play, although originally they were tight enough to require that they be driven in place.

It is extremely difficult to place an object as rigid as the slab on four supports and be certain that each support carries a fair share of the load. Repeated trials showed that a difference of as little as two- or three-thousandths of an inch in the height of one support was sufficient to cause the slab to rock and produce an impact which would result in a pattern similar to the one shown in figure 13. With the help of the company which manufactured the load cells and weighing kit, a system was developed whereby the static load of the floating slab on each cell could be determined. This made it possible to obtain approximately the same dead weight on each load cell.



WEIGHT	2 ND AXLE	1 ST AXLE
FROM PATTERN	22,000 LB.	15,000 LB.
CORRECT	12,700 LB.	12,700 LB.

Figure 13.—Weight pattern for a dual axle traveling 53 miles per hour, with the floating slab not resting uniformly on the load cells (oscilloscope sweep at high speed).

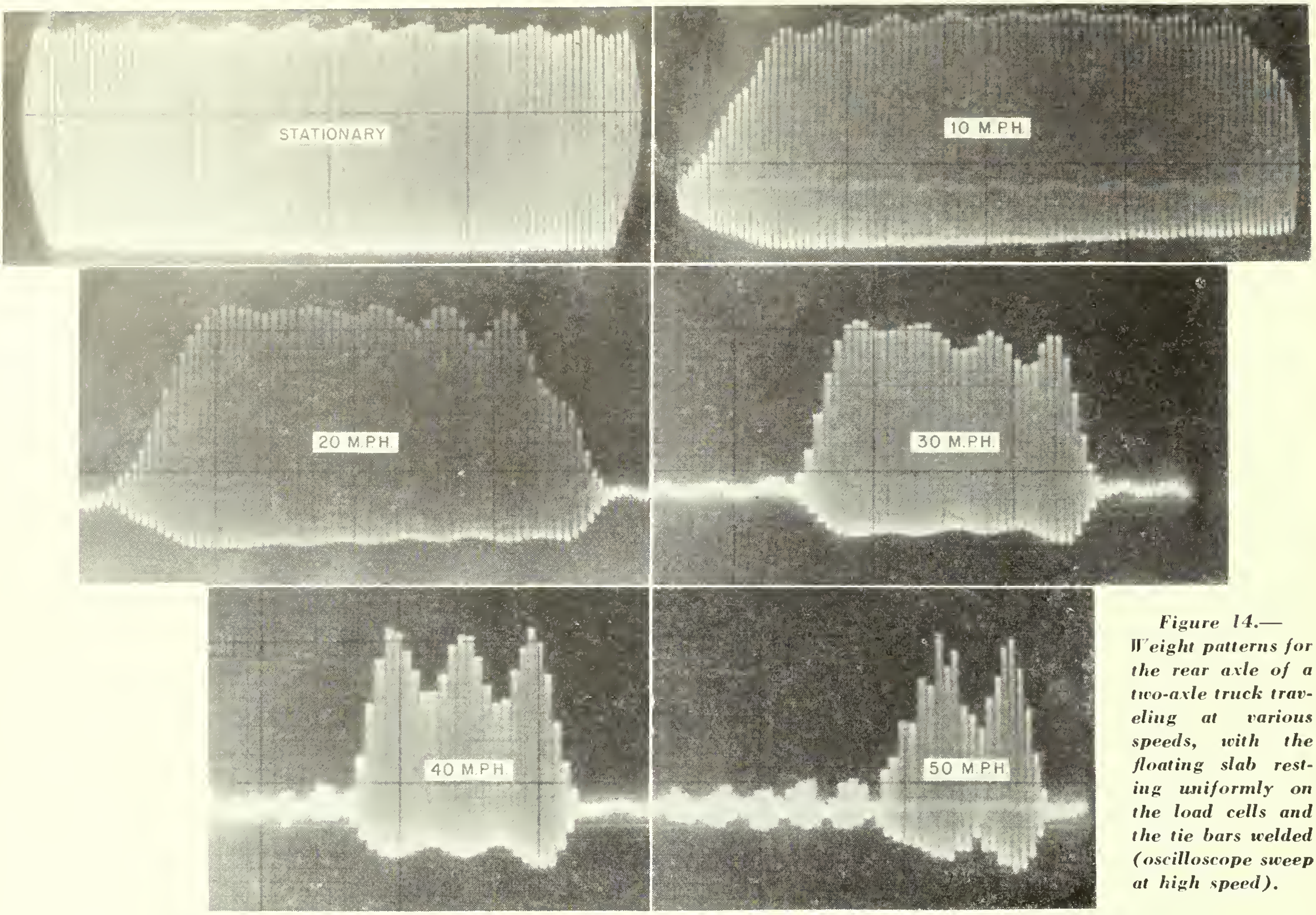


Figure 14.—Weight patterns for the rear axle of a two-axle truck traveling at various speeds, with the floating slab resting uniformly on the load cells and the tie bars welded (oscilloscope sweep at high speed).

Weight adjustments were made by grinding the bearing surface on top of the load cells rather than by the use of shims.

After the bearing points of the floating slab on the load cells had been adjusted so that each cell carried approximately the same portion of the total weight of the slab, which was about 5,000 pounds, and the horizontal tie bars had been welded, patterns similar to those shown in figure 14 were obtained. All of the separate patterns shown in figure 14 were made by the rear axle of a single-unit truck. Other patterns made by dual axles were similar, and none had the extremely ragged appearance of the pattern shown in figure 13. There are, however, several characteristics of these patterns which as yet have not been explained.

It may be noted that while the truck was stationary the pattern has a scalloped appearance at the top. This same repetition rate of the scallops is found in the pattern of weights of moving vehicles and appears to remain constant regardless of the speed of the vehicle. It apparently occurs at a frequency of about 60 cycles per second, corresponding with the frequency in the oscillation of the alternating current used to operate the electrical equipment. The magnitude of the peaks and valleys, however, increased with an increase in the speed of the vehicle. It may seem that the recorded load should be determined by using an average of the magnitude of the vertical lines at the peaks and at the valleys. This was not the case, however, since it can be seen from figure 14 that the length of the vertical lines at the peaks of all the patterns remains fairly constant regardless of speed, being about 16 divisions of the scale in all cases.

It is interesting to note that the zero line immediately to the left of the weight pattern increases in width with an increase in the speed of the vehicle. This increased width is undoubtedly caused by vibrations in the slab immediately after an axle had passed over it. It is also interesting to note that the peaks caused by the vibrations of the slab are four or five oscillations apart, which indicates that the frequency of vibration of the slab was 80 to 100 cycles per second. This frequency does not correspond with the frequency of the valleys and peaks in the load pattern.

Dual-Axle Weight Errors Continue

With the floating slab resting uniformly on the four load cells and with the tie bars welded in position, a series of tests was made of the accuracy of the scale by weighing vehicles as they passed the study location. Under normal operation about half of the trucks passing the location have all tires on the 10-foot platform. To increase the data obtained in a given period and eliminate the need of weighing trucks (at the lever-system scale) which did not have all tires on the electronic scale, rubber cones were placed on the center line of the pavement used by southbound vehicles. Most of the weight recordings were then usable data for comparison with weights obtained at the lever-system scale. Any trucks passing with one or more tires off the scale were indicated by a transverse placement detector and their records eliminated from further consideration.

During the first of a series of tests employing vehicles that normally passed the study location, trucks were slowed to 10 miles per hour for a short period and to 20, 30, and 40 miles per hour for other periods to obtain a range of speeds. During the final period of the test, the trucks were permitted to operate at their normal speeds. Figure 15 shows results of these tests.

In figure 15, the weight of each axle as shown by the oscilloscope pattern of the electronic scale is plotted against the actual weight of each axle as recorded at the lever-arm scale. When the plotted point falls on the 45-degree line, there is no difference between the two weight recordings. For some of the axles the plotted points fall on or near the line, but in other cases, especially for some of the dual axles, the plotted points fall off the 45-degree line a distance which represents more than 2,000 pounds. There is, however, fair similarity between the recorded weights at the two scales.

Based on the weights indicated by the lever-arm scale, table 3 shows the variation in the average error of the electronic scale for various vehicle speeds. The difference between the weight of an individual axle as recorded by the electronic scale while trucks were moving at their normal speeds and its weight as recorded statically on the lever-arm scale was, on an average, about 600 pounds for single axles and for the first axle of dual axles. The average error for the second axle of dual axles was 2,300 pounds. On a percentage basis, these errors amount to 6.5 percent for single axles, 4.4 for the first of dual axles, and 16.7 for the second of dual axles. At other speeds, the errors in some cases were higher than at normal speeds and in some cases lower, there being no consistency in the effect of speed.

The errors shown in table 3, which was just discussed, are average values for individual axles. Some of the weights recorded by the electronic scale were higher and some were lower than the weights as recorded at the lever-system scale. If the cumulative weight of all trucks recorded at the one scale is compared with the cumulative weight of the same trucks at the other scale, the errors are much lower, as shown in table 4. For example, table 4 shows that if it is desired only to obtain the total tonnage on the highway during the study period, the results obtained by the electronic scale would, on an average, be

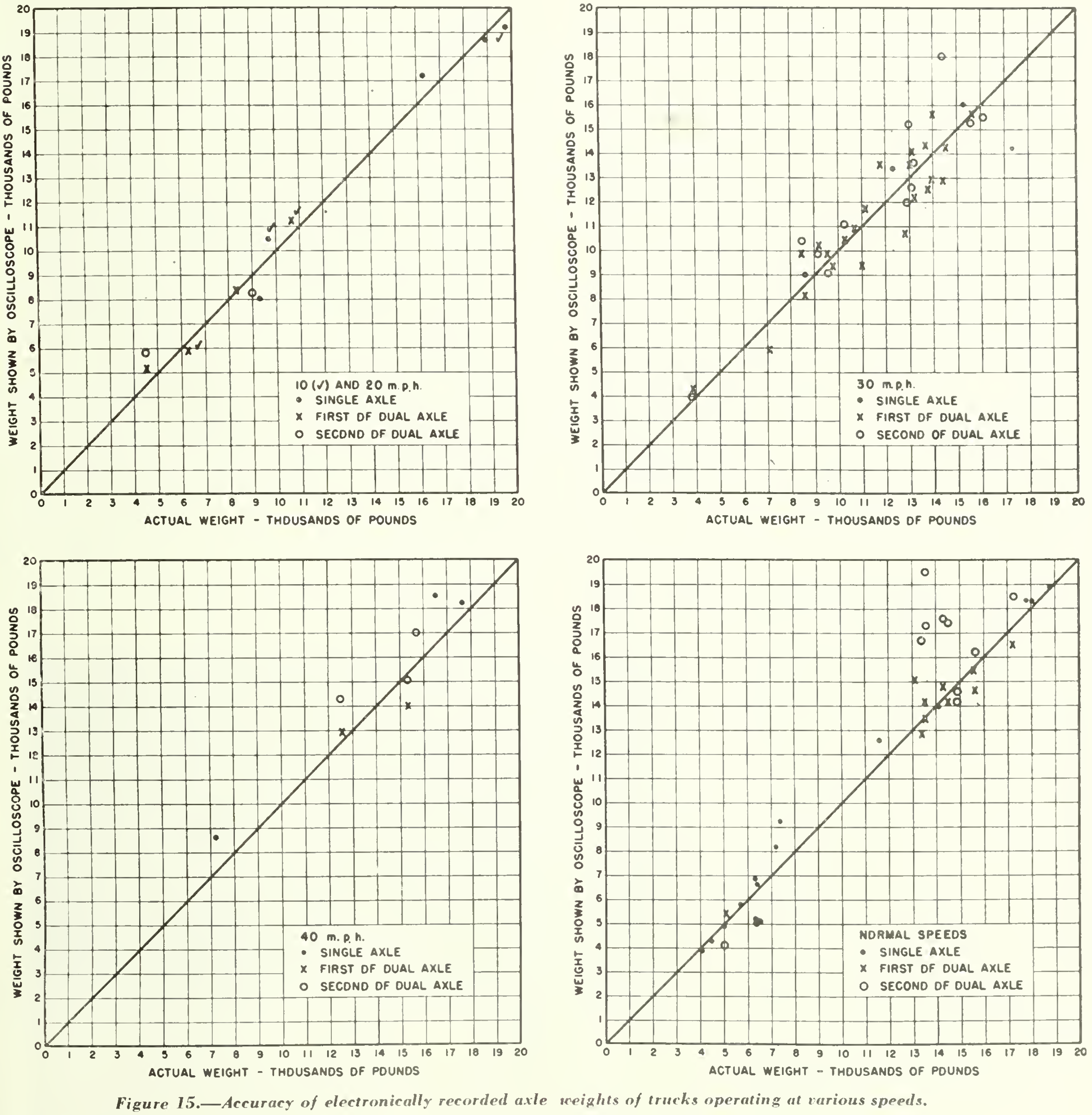


Figure 15.—Accuracy of electronically recorded axle weights of trucks operating at various speeds.

Table 3.—*Average absolute errors in recorded axle weights for trucks at various speeds*[1]

Speed of trucks	Error in pounds			Error in percent		
	Single axles	Dual axles		Single axles	Dual axles	
		First	Second		First	Second
10 m.p.h.	500	450	....	3.5	5.3	....
20 m.p.h.	915	325	1,025	5.6	5.1	16.0
30 m.p.h.	1,350	878	1,050	9.1	7.4	9.0
40 m.p.h.	1,333	850	1,100	9.8	6.1	7.6
Normal	612	600	2,300	6.5	4.4	16.7
Weighted average	815	755	1,517	6.9	6.3	12.2

[1] Based on total of 96 axle weights.

Table 4.—*Average algebraic error in recorded weights for trucks at various speeds*

Speed of trucks	Single axles		Dual axles	
	Lb.	*Percent*	*Lb.*	*Percent*
10 m.p.h.	+300	2.1	+50	0.6
20 m.p.h.	−5	.0	+675	10.1
30 m.p.h.	−250	1.7	+275	2.4
40 m.p.h.	+1,333	9.8	+258	1.8
Normal	+82	.9	+905	7.1
Weighted average	+166	1.5	+482	4.2

166 pounds higher for each single axle and 482 pounds higher for each dual axle than the results at the lever-arm scale. The total weight of all single axles as recorded by the electronic scale with the trucks in motion was 1.5 percent higher than their static weight recorded at the conventional lever-arm scale. The corresponding figure for dual axles was 4.2 percent.

High Truck Speeds

The normal speeds of trucks were higher at the electronic scale than on the average highway. Figure 16 shows that 74.3 percent of the trucks exceeded 45 miles per hour and 40 percent exceeded 50 miles per hour, with 2 percent in the 65–70–mile range. The average speed was 48.4 miles per hour. These are the speeds that the trucks were traveling when their weights were recorded by the electronic scale, except during special tests when the trucks were purposely slowed by special signs and police-officer control.

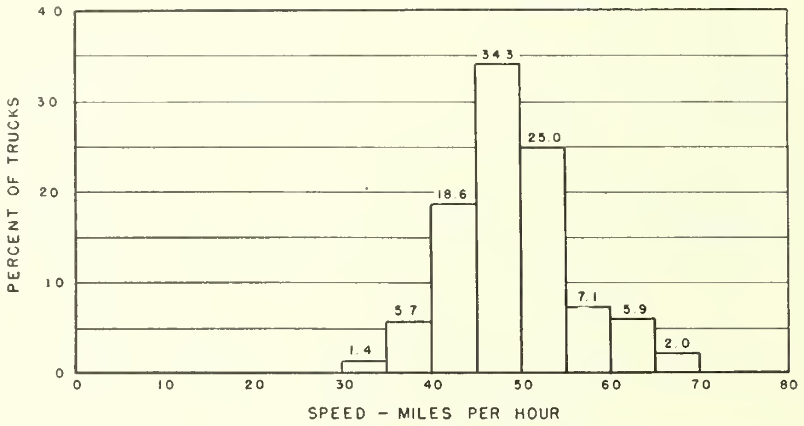
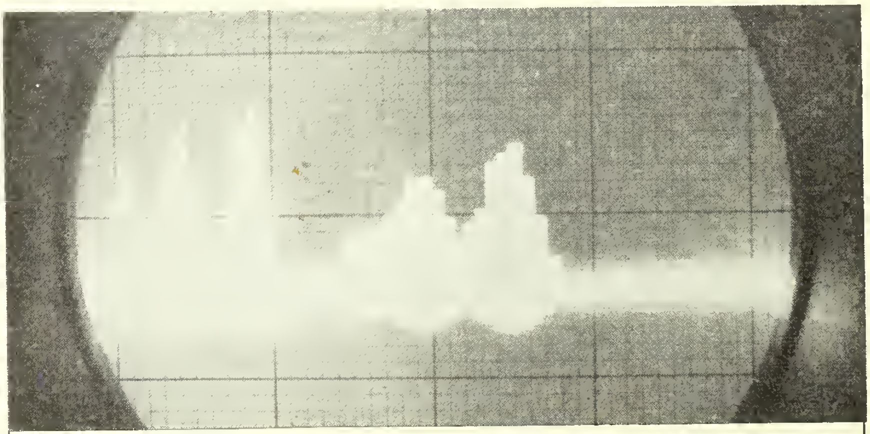


Figure 16.—Normal operating speeds of trucks at the electronic scale location.

Vibrations in the slab appeared to be causing erroneous weight recordings for axles that crossed the scale immediately behind other axles. These vibrations were the aftermath of the suddenly applied wheel loads. Support by an additional load cell placed under the center of the slab would have reduced the bending moments produced by the loads and probably reduced the magnitude of the vibrations. This, however, would have required changes in the design and calibration of the weighing kit since it had already been modified from a single-cell unit to one for use with four load cells (but not five).

As an alternative to an additional support, vertical bolts extending through the floating slab and securely anchored to the bottom of the pit were installed at the center and at each end of the floating slab. Tightening the nuts on the bolts against the top of the slab permitted the load cells and the slab to be preloaded. Tension up to 6,000 pounds could be applied to each of the three bolts. Preloading the slab would change its vibration period, thus permitting additional data to be obtained for a study of the vibration problem.

After a load had been applied with the bolts, the weight-recording instruments were adjusted to show a zero load with no truck on the scale. With a truck axle of a known weight on the scale, however, the weight as shown by the scale dials was less than the actual weight of the axle by the amount that the tension in the bolts was reduced due to the additional bending of the slab. For example, a truck having a weight of 18,000 pounds when there was no tension on the bolts showed an indicated reading of 14,600 pounds on the dials when there was an initial load of 4,200 pounds on the center bolt and no load on the end bolts. Likewise, with 5,000 pounds on the center bolt and 6,000 pounds on each end bolt, the dials showed a reading of only 10,400 pounds for the 18,000-pound axle weight. It was therefore necessary to calibrate the scale each time that a change was made in the tension placed in the bolts to preload the slab.



2ND AXLE	1ST AXLE	WEIGHT FROM PATTERN
14,500 LB.	11,200 LB.	MAXIMUM PEAK
12,600 LB.	9,100 LB.	CENTER PEAK
9,200 LB.	9,200 LB.	CORRECT

Figure 17.—Weight pattern of a dual axle traveling 45 miles per hour, with no preloading of the floating slab.

Preloading Improved Accuracy

Table 5.—*Effect of preloading of floating slab on average absolute errors in recorded weights of individual axles*

Load on floating slab	Single axles		Dual axles					
			First			Second		
	No.	Error	No.	Error		No.	Error	
		Lb.	*Percent*		*Lb.*	*Percent*		*Lb.*	*Percent*
No preloading	45	1,213	14.3	13	1,442	15.3	13	2,246	23.9
4,200 pounds on center bolt	17	767	7.2	6	592	4.9	6	928	7.2
5,000 pounds on center bolt; 6,000 pounds on each end bolt	29	625	5.5	10	790	6.0	11	1,140	8.7

Table 6.—*Effect of preloading of floating slab on average algebraic error in recorded total weights for all axles*

Load on floating slab	Single axles		Dual axles						
				First			Second		
	No.	Error		No.	Error		No.	Error	
		Lb.	*Percent*		*Lb.*	*Percent*		*Lb.*	*Percent*
No preloading	45	686	8.1	13	242	2.6	13	1,731	18.3
4,200 pounds on center bolt	17	21	.2	6	541	4.5	6	728	5.7
5,000 pounds on center bolt; 6,000 pounds on each end bolt	29	63	.6	10	−400	3.0	11	−195	1.5

The preloaded scale was calibrated by recording the static weights of several axles on both the electronic and lever-arm scale immediately prior to each test in which vehicles in motion were weighed. This procedure was necessary to eliminate the effects of temperature changes which affect the amount of tension in the bolts.

Tests following the addition of the bolts showed that the preloading produced smoother oscilloscope weight patterns and a greater accuracy in the weighing of vehicles in motion. Figure 17 shows the weight pattern recorded for a dual axle on a truck traveling 45 miles per hour when the slab was not preloaded. Figure 18 shows the same information for another truck, also traveling about 45 miles per hour, when the slab was preloaded 2,500 pounds by the center bolt.

In figure 17, each division of the vertical scale represents 1,000 pounds. The peak of the pattern for the first axle represents a load of 11,200 pounds, or 2,000 pounds more than the correct weight of the axle. The peak of the pattern for the second axle represents a load of 14,500 pounds, or 5,300 pounds more than the correct weight of the axle. Examination of the pattern shows that the correct weights of the axles are represented by the second peak in the pattern for the first axle, and by the third peak in the pattern for the second axle. This relation did not always hold true, however, and while it was generally possible for someone familiar with the shape of the patterns to estimate the weight of axles with a high degree of accuracy by using a little judgment, a satisfactory scale must show consistent results without involving personal opinion.

The shape of the pattern in figure 18 and other patterns made with the slab preloaded by applying tension on the bolts was generally less erratic than the patterns made without preloading the floating slab. All, however, were not as uniform as those of figure 18, nor did the weights correspond as closely with the correct weight of the axles. The patterns continued to show peaks of the same frequency regardless of the load on the bolts, apparently proving that the major peaks and valleys were not caused by vibrations of the slab or by impact loads produced by the trucks.

The effect that preloading had on the accuracy of the scale for weighing trucks traveling at their normal speeds is shown by figure 19. Prior to the tests during which these data were obtained, the bearing plates of the load cells were not adjusted so that each cell carried the same portion of the dead weight of the slab. The adjustment was omitted because it was one purpose of the test to determine whether preloading of the slab and cells would eliminate the need of such an adjustment. The results as shown by the upper chart in figure 19 are therefore more erratic than those of figure 15, which were obtained with the best possible adjustment of the bearing plates.

A study of the results shown in figure 19 indicates that there was a definite improvement in the accuracy of the scale when the slab was preloaded. A few erratic results were obtained, however, especially for the second axle of the dual axle shown in the lower left-hand chart of figure 19 for which a weight of 10,800 pounds was recorded while its correct weight was 17,800 pounds. It is believed that in this case one tire of the axle was not on the scale as the truck passed the study location. This would have been called to the attention of the scale operator had the lateral placement detector been in use during this test. Such an occurrence was so rare after the rubber cones were placed on the lane line that use of the placement detector was considered unnecessary.

The average errors of the electronic scale with and without preloading of the slab are shown in tables 5 and 6. In table 5, the average errors in the recorded weights of individual axles are shown. Preloading the slab by placing either a tension of 4,200 pounds on the center bolt or 5,000 pounds on the center bolt and 6,000 pounds on each of the end bolts reduced the magnitude of the errors, measured in pounds, by an average of about 50 percent. Measured as a percentage of the actual weight, preloading the slab cut the errors in the recorded

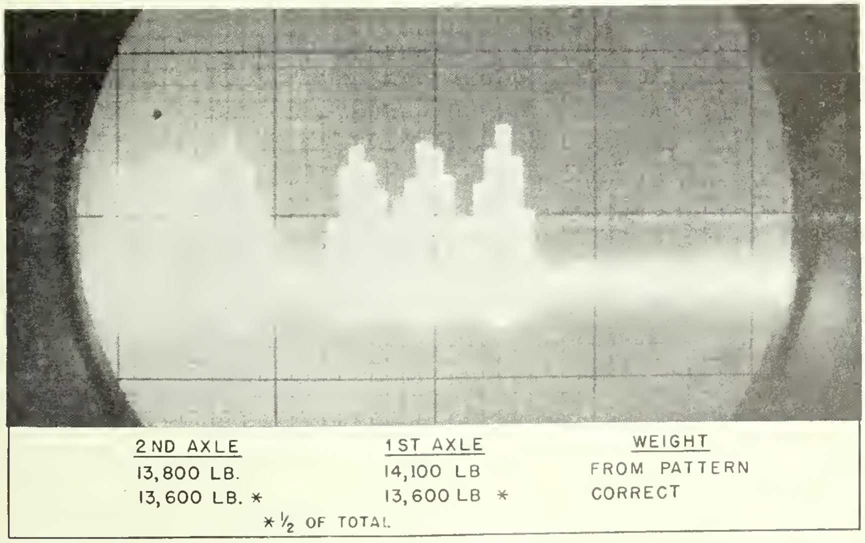


Figure 18.—Weight pattern of a dual axle traveling 45 miles per hour, with preloading of 2,500 pounds at the center of the floating slab.

weights of individual axles by at least half and in most cases by considerably more than half of what they were without preloading the slab.

Table 6 shows that when the slab was preloaded the total weight of all axles recorded while the trucks were traveling at their normal speeds was remarkably close to their total static weight. With a tension of 5,000 pounds on the center bolt and 6,000 pounds on each end bolt, the cumulative errors were plus 0.6 percent for the single axles, minus 3.0 percent for the first of the dual axles, and minus 1.5 percent for the second of the dual axles. These are well within the limits of precision necessary for such highway-planning purposes as a determination of gross tonnage of trucks traveling over a highway.

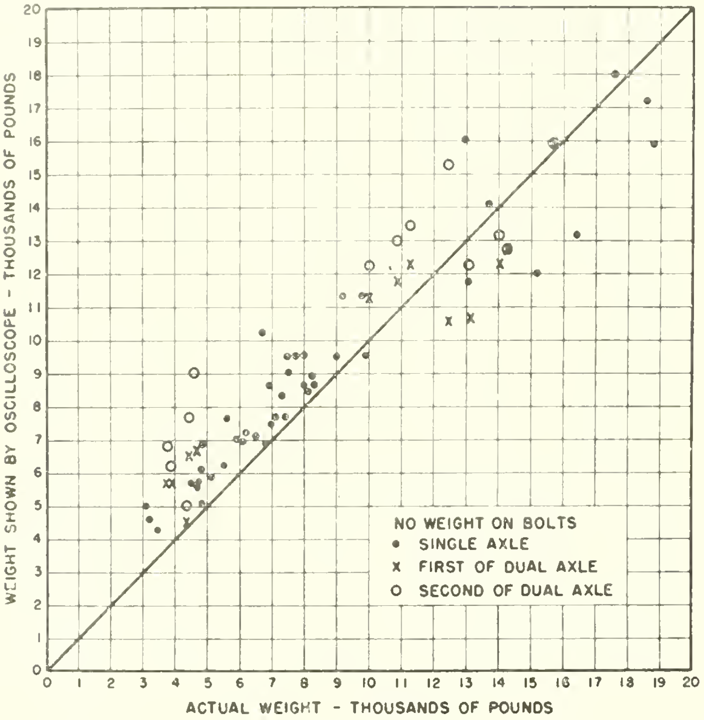
A factor which may have resulted in charging errors to the electronic scale improperly, throughout these studies, was the assumption that both axles of a dual axle carried the same weight whenever separate readings were not taken at the lever-system scale. For 63 percent of the dual axles, the second axle weighed more than the first axle, according to the lever-arm scale. The reverse was true for the other 37 percent. Table 7 shows the magnitude of the differences in weight between the two axles of the duals. The difference in four cases was about 4,000 pounds. Since separate weights were obtained at the lever-system scale for less than 50 percent of dual axles weighed at the electronic scale, the assumption that equal weights were carried on both axles of a dual axle when they were not weighed separately probably caused many of the major indicated errors of the electronic scale.

Table 7.—*Distribution of weight differentials between the two axles of duals*

Difference in weight	Number of dual axles
Pounds	*Percent*
Under 500	30.6
500 to 999	18.4
1,000 to 1,999	32.6
2,000 or more	18.4

Other Improvements Tried

While it is realized that the method used for preloading the slab would not be satisfactory for a practical scale because frequent recalibration would be required, the method did provide a means of studying the causes of inaccuracies. For a practical installation, additional load cells to measure the tension in the bolts should be connected in parallel with the present cells. This would eliminate the need for recali-



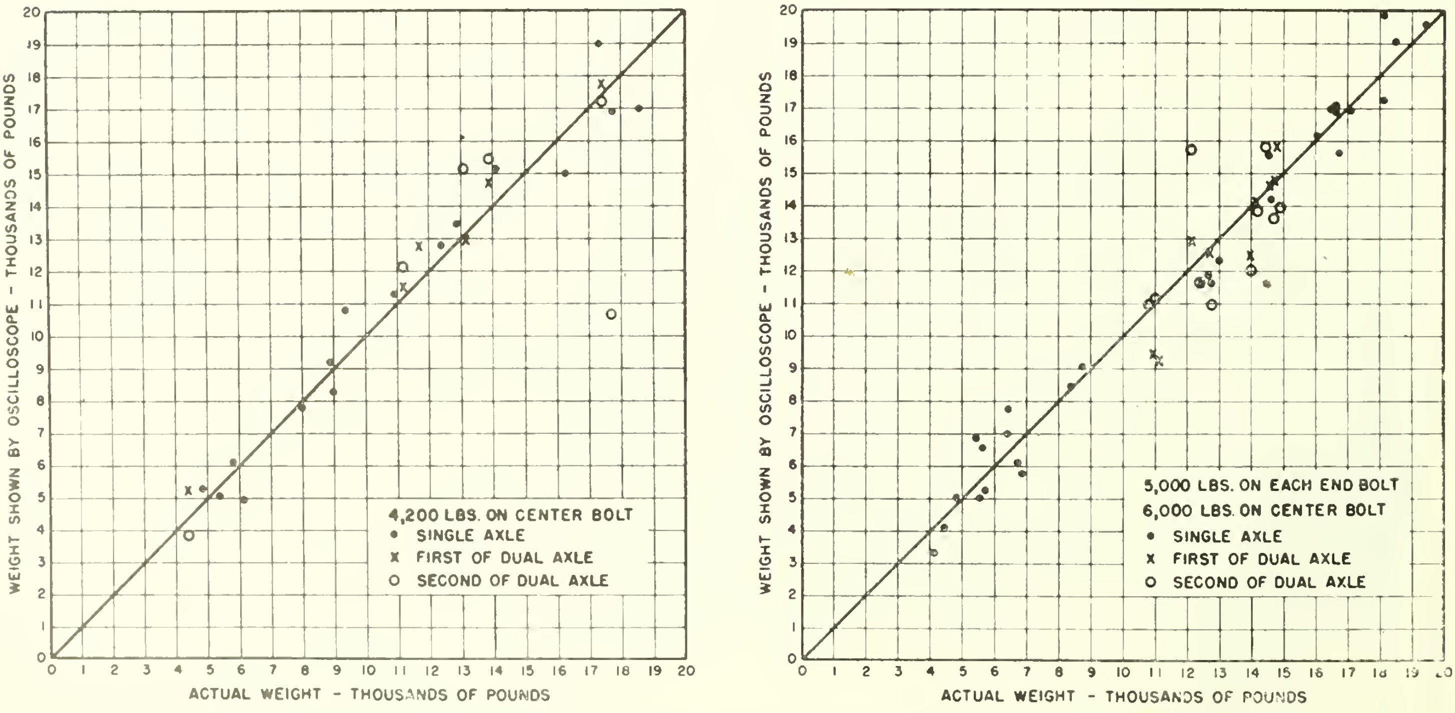


Figure 19.—Accuracy of electronically recorded axle weights, with no preloading of the floating slab (above), with preloading at the center, and with preloading at the center and both ends.

brating the instrument with changes in temperature.

As a further effort to improve the scale's accuracy, the electrical circuits were changed to include an integrator. Basically, the purpose of the integrator is to smooth the pattern by reducing the magnitudes of the valleys and peaks, and to change the behavior of the oscilloscope spot so that the load deflections are only in the one direction. This makes it possible to calibrate the screen so that each horizontal line represents a certain load, thereby eliminating the need for measuring the total length of the ordinate of a weight-pattern. Figure 20 shows a calibration pattern using the integrator, and figures 21 and 22 show typical weight patterns for trucks when the integrator was used. The accuracy of the scale using the integrator was not always as good, however, as figures 21 and 22 indicate. The results obtained during its use, as shown by figure 23, are too limited to use as a basis for determining whether or not the integrator helped to improve the accuracy of the scale.

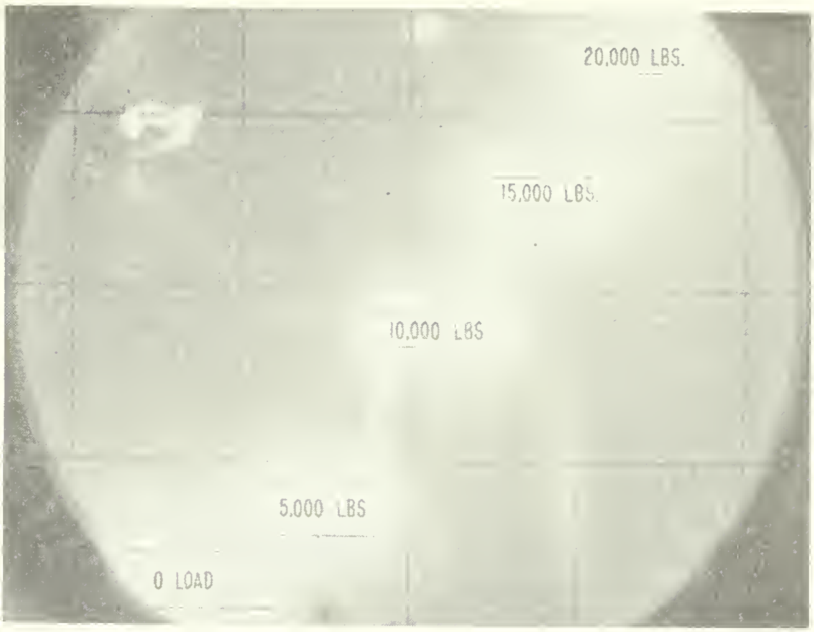
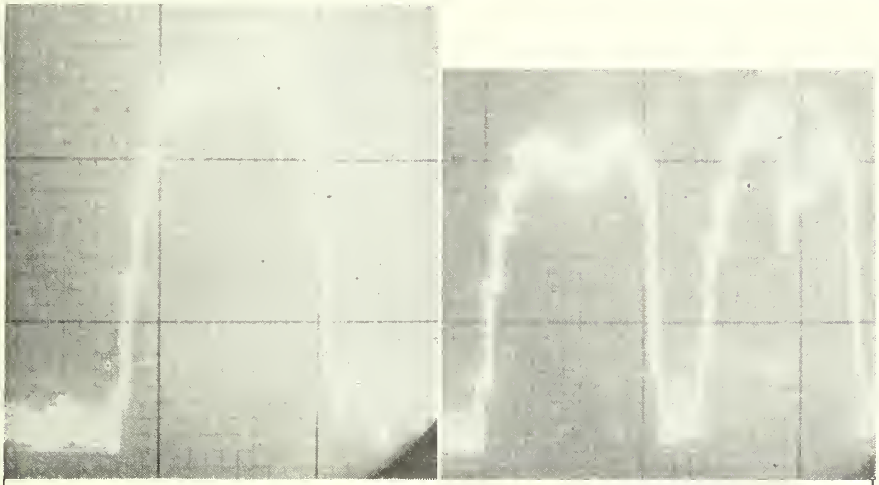


Figure 20.—Weight calibration pattern with an integrator used in the circuit.



SINGLE AXLE	WEIGHT	DUAL AXLE FIRST	DUAL AXLE SECOND
14,000 LB.	FROM PATTERN	12,700 LB.	13,550 LB
14,200 LB.	CORRECT	13,200 LB. *	13,200 LB *

* ½ OF TOTAL

Figure 21.—Weight patterns for a single axle and a dual axle, using the integrator (oscilloscope sweep at high speed).

In the latest change made on the scale, an oscillograph was substituted for the oscilloscope as the recording instrument. The advantages or disadvantages of this change cannot be determined without further tests. The oscillograph does, however, have one important advantage—a permanent record is obtained immediately without the need of taking a picture or developing a film. The type of record made by the oscillograph is shown in figure 24, and the data recorded during its use are shown in figure 25. Since the oscillograph had not been calibrated to show the load in pounds, figure 25 is useful only to show the uniformity of the results. They appear to compare favorably with results obtained with the oscilloscope.

Speed and Axle Spacings Measured

The equipment used for electronic weighing also permits the recording of vehicle speeds and axle spacings at the same time. Figures 26 and 27 show the accuracy in axle spacings obtained with the electronic weighing equipment.

More accurate measurements of axle spacings were possible after the square-wave generator was available to insert time marks on the screen than without the generator. The average error in measuring axle spacings was 0.5 foot without using the generator and 0.4 foot with the generator. These figures are based on the assumption that the manual measurements of axle spacings were correct to the nearest tenth of a foot. It is highly probable, how-

Figure 22.—Weight pattern for a three-axle truck, using the integrator (oscilloscope sweep at normal speed).

AXLE NUMBER	1	2	3
WT FROM PATTERN	4,600	10,600 LBS	9,350 LBS
CORRECT WEIGHT	4,600	10,680 "	9,360 "

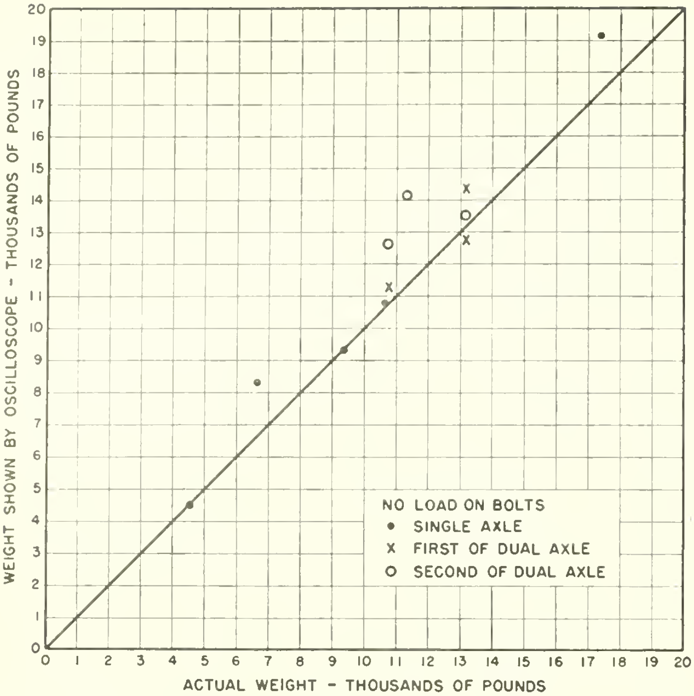


Figure 23.—Accuracy of electronically recorded axle weights, using the integrator.

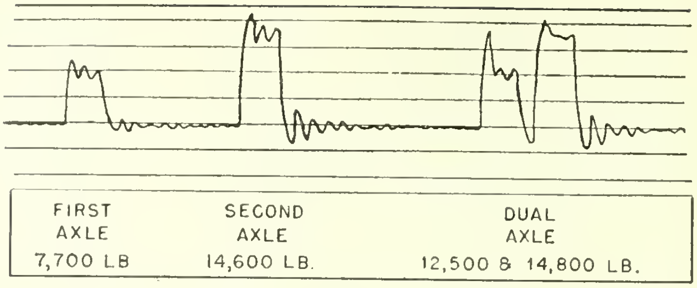


Figure 24.—Oscillograph recording of axle weights of a four-axle tractor-semitrailer.

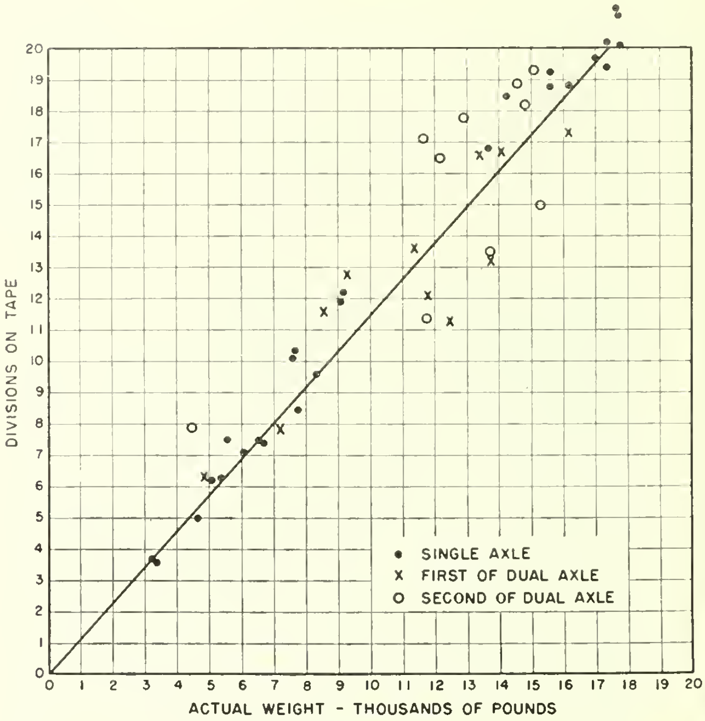


Figure 25.—Uniformity of oscillograph recordings of axle weights.

ever, that the results obtained electronically are more accurate than those obtained manually. Great care was not taken in the manual measurements because it was not expected that the electronic measurements made while the trucks were traveling at their normal speeds would be accurate to within less than one foot.

Design Improvements

The results and observations presented in this progress report are those that have been obtained to date. The scale is now being modified both structurally and electrically to embody several of the features that the tests thus far have shown to be desirable. Electronic improvements have already been discussed in the description of the circuitry.

The major change being made in the structural design involves the location of the load cells. In the original design, the load cells were nearly 10 feet apart in the transverse direction of the roadway. The tread width of the heavier trucks is approximately 8 feet, measured from outside to outside of tires. The centers of the loads transmitted to the pavement by the two sets of dual tires on one axle are thus about 6 feet apart.

Locating the load cells so that they are directly under the tires should practically eliminate the bending moments in the slab and the resulting vibrations. In the modified design, the load cells are being placed 6¼ feet apart.

Tie bars to prevent any horizontal movement of the floating slab are also being located at the surface of the pavement. These will supplement those already installed, which are about 15 inches below the surface. There are other changes that would be desirable but which cannot be incorporated in the present scale without rebuilding the entire pit and slab. For example, three-point suspension, which is entirely feasible, might ultimately be adopted should too-frequent adjustment of the bearing plates in a four-point suspension system be found to be necessary.

One essential improvement which has not been mentioned is the waterproofing of the load cells. The load cells used in these tests were housed in a supposedly waterproof jacket with a bag of silica gel to dehumidify the air within the jacket. On two occasions a cell was replaced because the jacket became filled with water. At the end of the tests, three of the jackets were filled with water and the fourth was damp. Moisture, of course, destroys the accuracy of resistance-wire strain gages. It is probable that some water in the cell jackets penetrated the water-resistant coatings of the strain gages and introduced undetected errors before a saturation of the gages, which caused obvious errors, made it necessary to replace the cells. Those now being installed are moistureproof in themselves, and should serve satisfactorily.

With the changes that are now being made, there is reason for confidence that the electronic scale will provide a means of obtaining data for highway planning and permit enforcement of legal limits at low costs and without inconveniencing the large majority of truck operators who load properly.

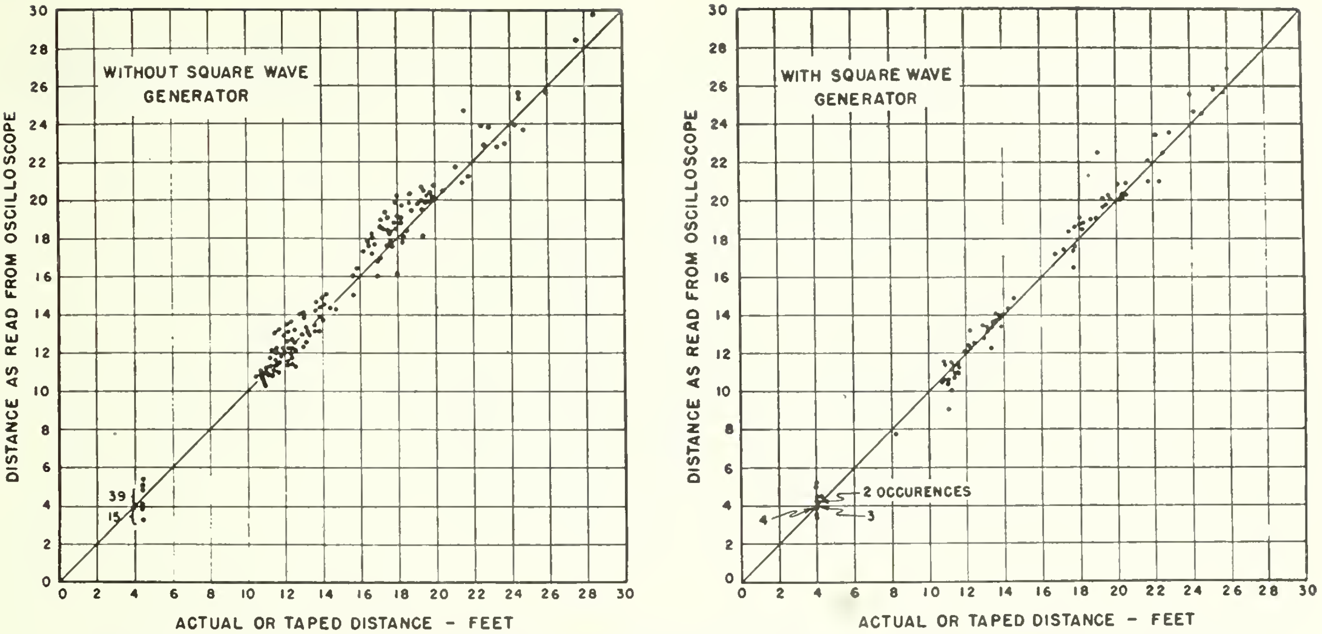


Figure 26.—Accuracy of electronically recorded axle spacings, with and without the square-wave generator.

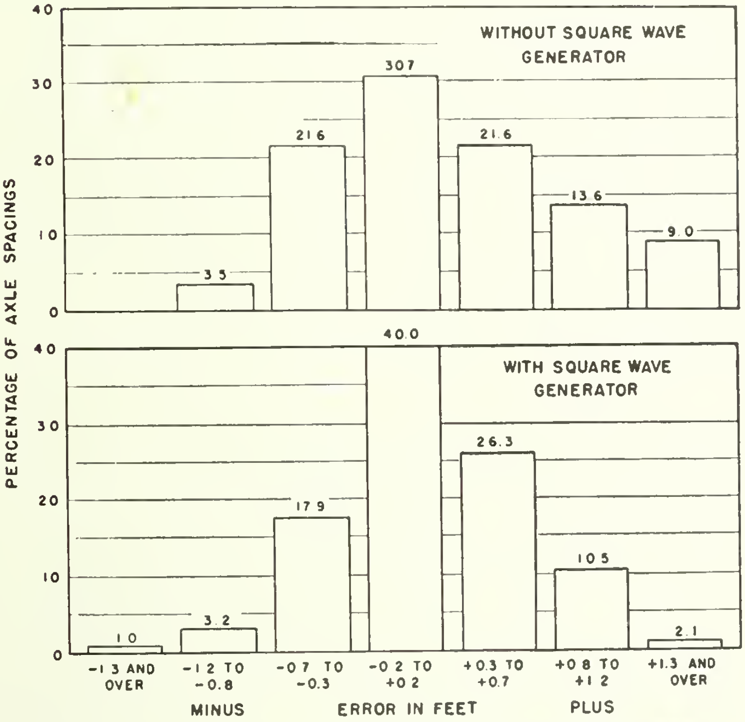


Figure 27.—Errors in electronically recorded axle spacings, at normal road speeds.

HIGHWAY STATISTICS, 1950

Now available is the Bureau of Public Roads' Highway Statistics, 1950, the sixth of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of highways, and highway mileage. Included for the first time is information concerning the financing of local streets. Highway Statistics, 1950 is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 60 cents a copy. The full series of the annual bulletins are available from the Superintendent of Documents, as indicated on the inside back cover of Public Roads.

Weathering Tests on SR-4 Strain Gages

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by E. G. WILES
Highway Physical Research Engineer

The use of SR-4 strain gages to determine strains in steel bridges is important in highway research. For such work it is desirable that the gages be attached and protected so that not only can they be used when first installed to record strain caused by loads, but also re-used for a similar purpose at some later time, possibly after a relatively long period of time has elapsed. Tests of several commercial materials, reported in this article, demonstrate that some of them will serve satisfactorily for cementing and weatherproofing the gages for at least 3½ years.

PREPARATORY to a test program in which strains caused by different types and magnitudes of vehicular loads on highway bridges are to be determined, two questions needed to be answered: Can SR-4 electrical resistance strain gages be maintained in working condition over a period of a few days when exposed to severe weather conditions; and can they be maintained over longer periods of time and reused when needed for additional tests on the same structure? In answer to these questions, the limited study reported here shows that SR-4 strain gages can be satisfactorily weatherproofed so as to record faithfully even after long periods of exposure to the weather.

Method of Test

Although other work has been done on this problem, the rather simple means adopted to determine the effectiveness of the gages and certain combinations of cements and weatherproofing materials have not been previously described in the literature.

In July 1948, 24 type A-3 SR-4 electrical resistance strain gages were installed in pairs on opposite sides, in the center of each of twelve ¼- by 2- by 14-inch cold-rolled steel bars. Different kinds of cements, weatherproofing materials, and installation techniques were used, varying from no covering over the gages to a rather heavy combination of coverings. The bars were then exposed to the weather.

At periodic intervals, to determine whether or not the gages were functioning, each steel bar was supported as a simple beam with a 12-inch span length and a fixed load was applied at the third points. Strain measurements were made at no load and again under the load condition, and the strain difference computed. Since the same loading was used throughout the tests, the strain difference remained relatively constant in all cases where the gages were performing properly.

Preparation of Specimens

The cold-rolled steel bars used as specimens were first cleaned and painted. Areas for the gages and for bearings for third-point loading and support were kept free from paint during painting operations by covering with pieces of cellulose tape. The tape was removed after the paint dried.

For identification purposes, bars were stamped A, B, C, etc., through L and, in order that bars might be kept in the same position when exposed, a letter T (for top) was marked on one side of each bar.

The gages were of the electrical resistance type and were purchased commercially. The gage wire was a cupro-nickel alloy and was cemented to a thin paper backing which made possible easy application to a test specimen. The effective gage length was thirteen-sixteenths of an inch. The resistance of each gage was approximately 120 ohms and the gage factor, which is the ratio of unit change in resistance to unit change in length, was determined by calibration by the manufacturer as 2.05 ± 1 percent.

Before installing the gages, the surface of the metal was sandpapered and then washed with carbon tetrachloride and acetone. Three kinds of cement were used to attach the gages to the steel bars, as follows:

Fourteen gages, on specimens A, D, F, G, H, K, and L, were attached by SR-4 cement purchased from the Baldwin-Lima-Hamilton Corporation as part of a kit of cementing materials.

Six gages, on specimens B, E, and I, were attached with Duco household cement purchased locally in Washington, D. C., in a tube.

Four gages, on specimens C and J, were attached with HO cement furnished by the Celanese Plastics Corporation. On specimen J, the heat from two 250-watt infrared lamps was concentrated on the gages for 20 minutes after gage application.

Pairs of stiff lead wires, 24-gage (0.020-inch diameter) phosphor bronze, were soldered to and at right angles with the gage pigtails. Pieces of adhesive tape were used as insulation between the lead wires and the steel bar. Additional tape over the wires held them parallel and against the surface of the insulating tape. Lead wires from the top and bottom gages extended out from opposite edges of the bar 1½ inches beyond and perpendicular with the edge.

After 4 days the gages were coated with the different types of weatherproofing material, except for gages on specimens A, B, and C, which were left uncoated.

Gages on specimens D and J were coated with approximately one-sixteenth-inch thickness of melted Petrosene A wax purchased from the Baldwin-Lima-Hamilton Corporation. On specimen J, over this wax coating was placed another coat of approximately one-eighth-inch thickness of Ozite B heated to 400° F. This material was purchased from the G. and W. Electric Specialty Company.

Gages on specimen E were coated with approximately one-sixteenth-inch thickness of vaseline from available stock. Gages on specimens F and G were covered with Dow-Corning silicone ignition sealing compound about one-sixteenth-inch thick: Gages on specimen F were covered with C24 compound and those on specimen G with C28 compound.

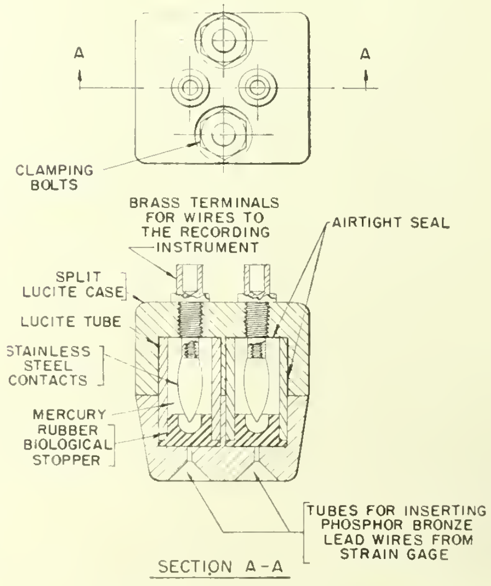

A
A
CLAMPING BOLTS
BRASS TERMINALS FOR WIRES TO THE RECORDING INSTRUMENT
AIRTIGHT SEAL
SPLIT LUCITE CASE
LUCITE TUBE
STAINLESS STEEL CONTACTS
MERCURY
RUBBER BIOLOGICAL STOPPER
TUBES FOR INSERTING PHOSPHOR BRONZE LEAD WIRES FROM STRAIN GAGE
SECTION A-A

Figure 1—Mercury connector.

Gages on specimen H were coated with approximately one-sixteenth-inch thickness of melted ceresin wax purchased on Government contract from the J. T. Baker Chemical Company. Gages on specimen I were first coated with about one-sixteenth-inch thickness of Ozite A heated to 400° F., followed by a wrapping of electrician's rubber tape and a second wrapping of friction tape. This top surface was then covered with another coating of Ozite A.

Gages on specimen K were covered with approximately one-eighth-inch thickness of Fenox No. 3005 furnished by the Bakelite Corporation. Gages on specimen L were covered with Short Stop, a dry-sealing insulator purchased from Buchanan Associates, Inc.

The materials used for adhesives and for weatherproofing in these tests were selected because they seemed to have possibilities for these purposes. Similar materials, produced by other manufacturers, might have been equally as effective as those covered in this report. Furthermore, some of the materials that were tested and found to be unsatisfactory were not produced for this use by their manufacturer; consequently their failure is not an indication of poor quality.

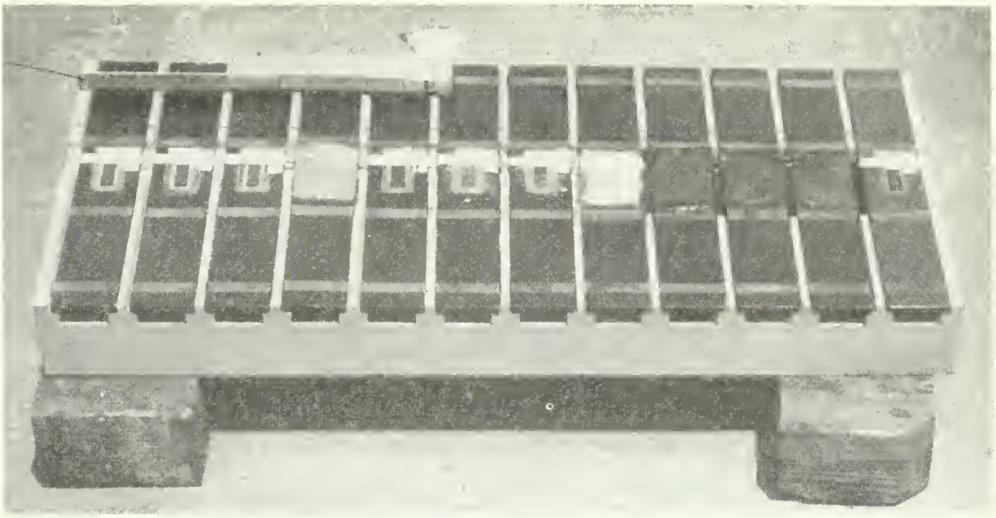


Figure 2.—Appearance of gages at beginning of weathering tests.

Mercury Connectors

Special mercury connectors were constructed to connect the phosphor-bronze lead wires to a suitable recording instrument. The connectors were a modification of an existing design[1] and were particularly suitable because of their low and constant contact resistance. The two principal modifications of these connectors were the use of stainless-steel bullet-nosed tips at the terminal ends in the mercury wells, to aid in making direct contact with the phosphor-bronze wires after they have been pushed into the wells; and the use of cross-slitted rubber biological stoppers as diaphragms, to hold back the mercury. No. 18 solid copper insulated wires connected the mercury connectors to the recording instrument. A drawing of the modified connectors, as used on these tests, is shown in figure 1.

The ends of the stiff phosphor-bronze lead wires connected to the gages and extending out from the edge of the bar are guided into the bottom of the tubes through the small inverted funnels. By holding the wires firmly and giving a slight push, the ends of the wires can be forced through the rubber biological stoppers and into the mercury wells. After readings are obtained the lead wires are withdrawn. The rubber diaphragms close behind the wires and prevent mercury from escaping.

Weathering Tests

The steel bars on which the gages were mounted were placed horizontally on a wooden rack, as shown in figure 2, and exposed to prevailing weather conditions on the flat roof of a building near Washington, D. C. One gage on each bar was exposed face up to the elements and the other gage of the pair was face down towards the roof. The rack was blocked up so that air could circulate freely under and around the gages. The conditions of exposure of pairs of these gages simulated those to which gages might be exposed when mounted on horizontal bridge members. Observations were made initially at intervals of a few days, later at weekly intervals, and at present the observations are being made at intervals of approximately a month.

Strain Measurements

On days when strain measurements were to be made, the rack full of bars was brought into the laboratory a few hours in advance in order that they might adjust themselves to laboratory temperature conditions. In no case were bars brought in when it was raining or when it appeared that rain was imminent. At first, unpainted places on the bars, left for the load and support bearings, were coated with grease to prevent rusting. The attempt at rust prevention was not successful, however, and after the first few weeks no further care was given these surfaces. This had no apparent effect on the test results.

Initially an SR-4 Wheatstone bridge control box was used to measure strains, but 40 days after the beginning of the tests a type K strain indicator was purchased and it has been used since.

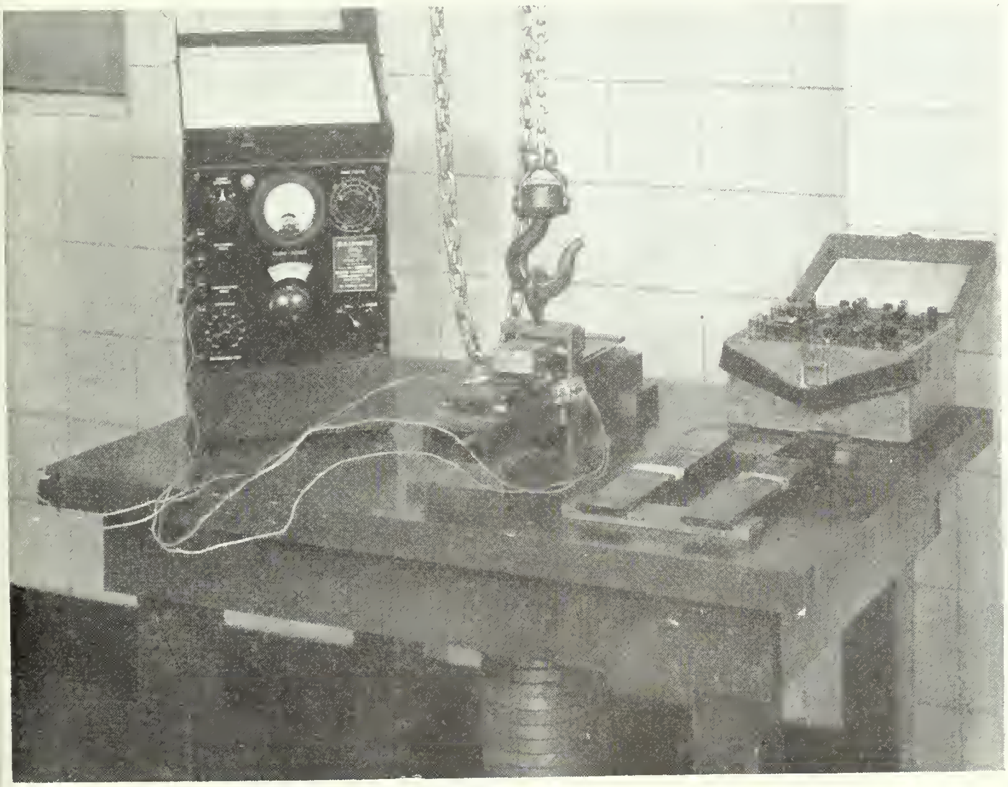


Figure 3.—Equipment for making strain measurements.

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[1] *Improved techniques and devices for stress analysis with resistance wire gages*, by W. V. Bassett, H. Cromwell, and W. E. Wooster. Proceedings of the Society for Experimental Stress Analysis, vol. III, No. 2, p. 76.

Table 1.—*Materials and service lives of SR-4 strain gages in weathering tests*

Specimen No.	Cement	Weatherproofing material	Service life	
			Top gage	Bottom gage
			Days	*Days*
A	SR-4	None	224	186
B	Duco	None	224	116
C	HO	None	224	116
D	SR-4	Petrosene A wax	1,300+	1,300+
E	Duco	Vaseline	441	441
F	SR-4	DC 4 ignition sealing compound C 24	186	580
G	SR-4	DC 4 ignition sealing compound C 28	224	818
H	SR-4	Ceresin wax	257	257
I	Duco	Ozite A, rubber tape, friction tape, Ozite A	1,300+	1,300+
J	HO	Petrosene A wax, Ozite B	1,300+	1,300+
K	SR-4	Fenox No. 3005	1,300+	1,300+
L	SR-4	Short Stop	25	1,300+

As previously mentioned, the bar was deflected as a simple beam under loads applied at the third points of the span length. Figure 3 shows the equipment used for applying load and for measuring strains. A total load of 212 pounds was supported by a chain hoist, until after the bar had been positioned on the supports, and an initial no-load reading obtained on the strain indicator. The load was then lowered until supported completely by the steel bar. A yoke moving in guides kept the third-point loading device and the suspended weights in position. The weights, attached by a hooked rod to the yoke, can be seen under the table. An opening in the center of the table gave sufficient clearance for the yoke, supporting the weights, to move freely. The type K indicator is at the left while a Leeds and Northrup type S testing set, used for measuring resistances of the gages, is shown at the right. More recently a Simpson model 260 volt-ohm-milliammeter has been used for a more rapid checking of the gage resistances.

In order to prevent any difference between the two mercury terminals from affecting the strain readings, the upper strain gages were always connected to one terminal while the bottom gages were always connected to the other. These mercury terminals, in turn, were always connected correspondingly to the same terminal posts on the strain-measuring equipment.

By having the gage on the tension side of the beam serve as one arm of a Wheatstone bridge and the gage on the compression side serve as an adjacent arm of the same bridge, temperature compensation was provided and the strain magnitude measured was the total for the two gages, or double that of either the tension or compression value. For the conditions described the combined strain measured was approximately 1,400 microinches. Normally, gages would record this amount of strain throughout their service life. Failure of gages was indicated by a marked change from this value.

If failure occurred in the top gage of a beam being loaded, the mercury terminal normally used to connect to it was connected instead to the top gage on another bar. This second bar was not loaded, but was placed adjacent to the loaded one and used solely to compensate for temperature changes. A similar procedure was followed in case a bottom gage failed. In these instances only one gage was deformed by the load and consequently the strain recorded was approximately 700 microinches.

Service Life

Now, after approximately 3½ years of exposure, it is of interest that pairs of gages on four bars and the bottom gage on one additional bar are still functioning satisfactorily. A tabulation showing the different types of cements, the weatherproofing, and the service life for all of the specimens is shown in table 1.

The six gages on three bars that had no special weatherproofing had surprisingly long service lives, from 116 to 224 days. The bottom gages failed first in each case.

The appearance of the top gages on these bars after failure to record strain is shown in figure 4. Moisture, corrosion, and temperature changes caused the gages eventually to peel away from the surface of the bar and no longer respond to deformation caused by load. They indicate the gage resistance of approximately 120 ohms with a very slight increase in strain under load because a small part of their effective gage length is still in contact with the bar.

Of the four pairs of gages still functioning satisfactorily, thus showing effective weatherproofing, Petrosene A wax and Ozite, either singly or in combination, were used on three pairs (on specimens D, I, and J). Heat is required to melt and apply both of these materials, Petrosene being a microcrystalline wax and Ozite a bituminous compound. For laboratory installations the use of these materials is not a problem. For field installations, however, heating the materials to proper temperatures and applying them to gages, especially on the underside of bridge members, might be difficult.

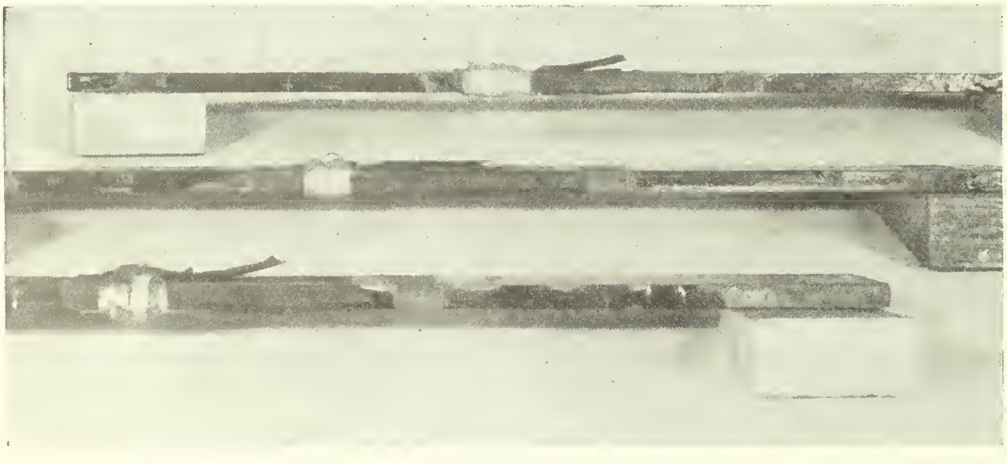

Figure 4.—Appearance of gages after failure on bars A, B, and C.

The other material that has proved to be effective in weatherproofing a pair of gages (on specimen K) is Fenox No. 3005, an electrical sealing compound. It is slightly tacky and adheres readily to the metal surface surrounding the gages. It is of puttylike consistency and can be applied with a spatula or the fingers and heating is not required. Exposure does not appear to have changed its properties.

The bottom gage on the beam (specimen L) that is still recording strain has a coating material different from any of the above. This material, known as Short Stop, forms a varnishlike coating, and has continued to give satisfactory weathering protection on the bottom gage, in spite of the fact that the top gage on this same bar was the first one to fail. It is a matter of conjecture as to whether this failure was inherent in the gage itself or caused by some deterioration of the weatherproofing material. For more positive results in tests of this kind more than one specimen of each type of weatherproofing should be included. However, it is indicated by these tests that SR-4 gages mounted on steel can be weatherproofed and will record satisfactorily after long periods of exposure to all of the usual weather conditions.

Acknowledgments

The author wishes to thank various individuals and companies for their helpfulness in giving information and, in some instances, samples of material for installing and weatherproofing strain gages.

Appreciation is expressed also to personnel at the David Taylor Model Basin, the National Bureau of Standards, and the Engineer Research and Development Laboratories, who furnished some information regarding installation and weatherproofing of gages.

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MISCELLANEOUS PUBLICATIONS

Bibliography on Automobile Parking in the United States.
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Bibliography on Highway Safety.
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Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.
Title sheets for PUBLIC ROADS, volumes 24, 25, and 26.

U.S. GOVERNMENT PRINTING OFFICE: 1952-951448

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 29, 1952

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$16,924	$22,548	$11,644	400.6	$5,339	$2,700	102.8	$20,512	$10,302	401.1	$48,399	$24,646	904.5
Arizona	1,246	7,513	5,426	179.6	1,130	806	13.1	7,701	5,334	118.7	16,344	11,566	311.4
Arkansas	8,370	7,323	4,045	292.3	6,284	3,107	216.4	12,290	6,341	333.5	25,897	13,493	842.2
California	6,452	53,020	20,492	204.6	12,722	6,531	38.6	66,593	31,775	241.4	132,335	58,798	484.6
Colorado	6,828	4,712	2,602	112.9	3,049	1,709	61.4	11,708	6,375	233.9	19,469	10,686	408.2
Connecticut	6,848	4,470	2,285	9.7	1,099	548	1.0	12,248	6,368	19.7	17,817	9,201	30.4
Delaware	2,221	6,385	2,693	31.7	329	234	7.0	3,199	1,594	31.2	9,913	4,521	69.9
Florida	9,034	11,090	5,714	192.5	8,914	4,723	173.7	16,360	8,140	310.4	36,364	18,577	676.6
Georgia	12,645	11,283	5,771	264.0	11,106	5,049	115.6	30,036	14,962	472.4	52,425	25,782	852.0
Idaho	7,134	9,970	6,240	269.8	2,703	1,719	79.3	3,162	2,069	73.6	15,835	10,028	422.7
Illinois	15,894	63,171	34,386	485.6	20,855	10,444	171.2	63,518	32,077	488.8	147,544	76,907	1,145.6
Indiana	18,212	33,770	17,470	174.9	9,260	5,114	72.2	17,744	8,982	195.4	60,774	31,566	442.5
Iowa	4,262	17,194	9,019	652.0	11,923	5,972	383.3	13,982	6,967	569.5	43,099	21,958	1,604.8
Kansas	11,867	11,495	5,598	1,125.8	3,176	1,565	262.0	16,079	8,414	639.2	30,750	15,577	2,027.0
Kentucky	9,323	15,052	8,058	218.2	7,405	3,673	185.6	11,279	5,693	159.0	33,736	17,424	562.8
Louisiana	7,106	16,587	7,942	86.2	8,613	3,905	81.9	20,165	10,102	169.4	45,365	21,949	337.5
Maine	4,562	5,923	3,333	30.0	1,742	867	10.6	7,308	3,707	62.8	14,973	7,907	103.4
Maryland	8,821	3,200	1,517	55.8	2,493	1,721	.7	9,517	4,443	25.3	15,210	7,681	81.8
Massachusetts	8,247	8,860	4,563	13.9	3,388	1,491	12.5	49,924	24,198	40.1	62,172	30,252	66.5
Michigan	7,097	36,550	18,239	542.1	8,865	4,444	140.4	51,949	22,439	275.0	97,364	45,122	957.5
Minnesota	9,214	14,584	7,484	1,405.3	5,240	2,613	291.5	20,641	11,309	638.7	40,465	21,406	2,335.5
Mississippi	10,870	10,364	5,352	336.4	3,410	1,672	121.6	18,456	9,610	466.9	32,230	16,634	924.9
Missouri	14,064	30,003	15,144	704.3	15,030	7,794	287.3	36,699	19,188	576.1	81,732	42,126	1,567.7
Montana	10,788	12,289	7,338	415.6	4,286	2,741	78.2	15,371	9,044	223.8	31,946	19,123	717.6
Nebraska	13,211	12,280	6,424	485.3	7,924	3,940	199.7	15,777	7,826	511.1	35,981	18,190	1,196.1
Nevada	7,116	2,756	1,996	83.4	1,972	1,650	44.6	3,266	2,708	134.8	7,994	6,354	262.8
New Hampshire	3,199	2,790	1,581	17.1	1,077	537	6.3	5,828	2,902	40.0	9,695	5,020	63.4
New Jersey	5,870	10,632	5,196	34.3	11,345	5,672	4.3	24,849	12,100	20.5	46,826	22,968	59.1
New Mexico	3,328	7,010	4,487	253.7	1,551	1,001	40.6	12,170	7,783	270.4	20,731	13,271	564.7
New York	48,444	72,364	37,858	144.1	26,218	12,002	141.5	90,690	41,475	277.9	189,272	91,335	563.5
North Carolina	8,662	22,773	11,074	548.9	7,007	3,229	110.2	19,557	9,823	376.3	49,337	24,126	1,035.4
North Dakota	3,416	11,939	6,149	1,253.9	5,887	2,939	551.5	7,689	3,885	630.2	25,515	12,973	2,435.6
Ohio	18,061	34,711	17,284	175.6	14,739	7,071	74.4	68,429	34,877	234.9	117,879	59,232	484.9
Oklahoma	9,789	9,256	5,295	137.9	7,387	3,866	118.1	17,870	9,532	172.5	34,513	18,693	428.5
Oregon	2,045	7,088	4,277	119.9	3,205	1,816	86.1	15,534	8,635	159.7	25,827	14,728	365.7
Pennsylvania	18,831	35,513	17,757	37.3	7,169	3,578	9.4	68,775	34,136	191.2	111,457	55,471	237.9
Rhode Island	1,576	5,713	2,856	43.0	2,153	1,077	13.2	17,817	9,163	12.5	25,683	13,096	68.7
South Carolina	5,173	12,338	6,639	387.9	2,916	1,505	149.6	12,592	6,284	257.9	27,846	14,428	795.4
South Dakota	1,849	10,633	6,129	827.6	4,057	2,444	253.8	12,282	7,062	661.3	26,972	15,635	1,742.7
Tennessee	8,405	12,258	5,746	339.7	11,512	5,486	215.1	25,270	12,032	292.4	49,040	23,264	847.2
Texas	25,672	6,145	2,950	241.2	12,231	6,157	453.8	53,921	27,964	658.5	72,297	37,071	1,353.5
Utah	3,928	5,184	3,933	93.5	2,976	2,205	78.6	4,185	3,009	55.8	12,345	9,147	227.9
Vermont	1,577	5,408	2,917	63.9	826	417	9.5	5,504	2,736	36.0	11,738	6,070	109.4
Virginia	11,545	15,789	7,819	331.3	7,345	3,699	156.1	21,082	10,510	336.3	44,216	22,028	823.7
Washington	9,108	6,045	2,354	110.1	2,896	1,192	24.6	20,092	10,462	108.6	29,033	14,008	243.3
West Virginia	6,939	11,944	6,032	115.3	2,778	1,388	28.2	11,058	5,524	116.8	25,780	12,944	260.3
Wisconsin	8,074	31,375	16,281	627.7	9,699	4,994	126.6	17,263	8,695	392.9	58,337	29,970	1,147.2
Wyoming	2,522	3,659	2,476	68.8	1,905	1,241	26.5	5,727	3,703	134.6	11,291	7,420	229.9
Hawaii	2,811	5,791	2,598	12.5	25	13	.1	11,656	4,662	33.9	17,472	7,273	46.5
District of Columbia	6,396	2,894	1,447	.7				3,252	1,819	.9	6,146	3,266	1.6
Puerto Rico	5,806	10,972	5,254	56.9	979	446	4.4	9,672	4,443	32.2	21,623	10,143	93.5
TOTAL	461,382	792,616	407,164	14,815.3	316,140	160,707	5,834.7	1,118,248	563,183	12,916.0	2,227,004	1,131,054	33,566.0

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JUNE 1952

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Barrier-line locations for no-passing zones are compared in this issue
(U S 66 near Lebanon, Missouri)

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

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The Effect of Barrier-Line Location at No-Passing Zones

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by CHARLES W. PRISK
Highway Engineer

In standard practice the barrier line at no-passing zones on two-lane highways is placed parallel and close to the roadway center-line marking. Missouri and Iowa, however, place the barrier line in the center of the lane from which passing is prohibited. The plan was developed in Missouri largely as an economy measure which has particular application on narrow pavement, for the painted stripe in this mid-lane location is not subject to constant wear by vehicle tires. A comparative study of the two marking types, conducted in cooperation with the Missouri State Highway Department on a 16-mile section of U S 66, is reported in this article.

No-passing zones on half the test section were marked with the Missouri standard, and on the other half the national standard was used. Two selected single-direction no-passing zones, one of each type, were studied intensively. A two-direction no-passing zone with Missouri markings was also studied. Information was collected on speeds and transverse placements of vehicles and on starting and finishing points of all passing maneuvers, approaching and within these zones.

Some of the traffic operation characteristics differed but little on the two types of markings. In critical conditions for transverse placement, however, particularly for vehicles traveling into a no-passing zone in the face of oncoming traffic, or where wide vehicles were involved, the advantage was consistently with the national standard marking. The national standard also showed favorable performance in a comparison of daylight and night driving, and in the extent to which drivers complied with the no-passing restriction. Obscurement of the mid-lane barrier line by preceding vehicles undoubtedly caused some drivers to encroach, without deliberate intention, on the no-passing zone area. Interviews with drivers leaving the test section revealed no decisive preference for either type of marking.

THE two-lane, two-way highway is the wheel horse of our transportation system. For economic reasons, it will undoubtedly persist as a popular type, despite the operating hazards inherent in its design. Warranted, therefore, are the many researches dedicated to improvements in the traffic capacity, safety, and other functional characteristics of this most common of all highway types.

Vehicle overtaking and passing actions on two-lane highways are essential to maintenance of reasonable capacities and necessary flexibility in traffic flow. Passings can normally be undertaken by the driver at any time he is assured that the left lane will be free of oncoming traffic throughout the time of his maneuver. The presence of opposing traffic that might interfere with passing is obvious when sight distances are adequate, but wherever alinement changes or other factors introduce a short sight-distance condition, the driver has no positive assurance that his passing can be completed without interference. This has led to rather general use of the no-passing-zone pavement marking which defines the limited section throughout which passing is not safe. According to reports of the President's Highway Safety Conference, 27 State highway departments are regularly marking no-passing zones, where needed, on more than 70 percent of the hard-surfaced mileage under their jurisdiction. The remaining States are similarly marking substantial portions of their mileage, and the totals are increasing each year. The value of this control measure is now well established.

Standardization of No-Passing-Zone Markings

What drivers see and understand has a profound effect on their driving actions and reactions, and what they do not fully see or understand often bears on their traffic mishaps. Much effort has been directed, therefore, toward standardizing all traffic-control devices, among them the marking design for no-passing zones. Many differences exist among the States. Since no-passing zones are locations of exposure to relatively high hazard, it is vitally important that all drivers quickly and accurately interpret the "line language" of pavement marking. Variations in color, in pattern, and in location of the barrier line with respect to the center-line marking subject motorists to possible confusion that may result in abnormal behavior or accident.

Though there is only partial agreement on certain details of the no-passing-zone marking design, practically all State and local jurisdictions place the barrier line a few inches to the side of the center-line marking to indicate that passing is prohibited from the lane on that side of the center line. This is the nationally recommended standard.[1]

Drastic exceptions to the national standard are found in Missouri and Iowa where, for a number of years, the barrier line has been positioned in the center of the driving lane from which passings are prohibited. This marking design was first adopted by Missouri to facilitate maintenance of a serviceable barrier line at a lateral position on the roadway where it would not be subject to the constant grinding and abrasive action created by heavy commercial tires. On narrow pavements, especially those with a lip curb, the problem of maintaining a serviceable barrier line near the roadway center was most pronounced. Placing the barrier line at the center of the driving lane produces the economy of longer effective life—in some cases, serviceable characteristics have been retained for as long as 3 years. Also given consideration in the Missouri decision was the relatively greater hazard of painting lines where the striping equipment has to straddle the center line rather than proceed entirely within one lane.

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[1] *Manual on Uniform Traffic Control Devices for Streets and Highways*, prepared by a joint committee of the American Association of State Highway Officials, the Institute of Traffic Engineers, and the National Conference on Street and Highway Safety. Published by the Bureau of Public Roads.

Missouri Site Studied

To evaluate the effect of the two types of no-passing zones on driving practices, the Missouri State Highway Department and the Bureau of Public Roads cooperated in a series of special traffic studies in that State during the summer of 1949. The highway selected was U S 66, a principal cross-country route with a high proportion of out-of-State traffic.

In the 16.5-mile test section chosen for study, approximately one-half the length was marked with the national standard no-passing-zone design; the other half with the Missouri standard design. Two single-direction no-passing zones as nearly alike as possible, one with the national standard marking and one with the Missouri marking, were studied intensively. A two-direction no-passing zone with Missouri markings was also studied.

Three types of information were assembled. First, and perhaps most significant, was a vehicle speed and transverse-placement study, during which data were obtained for more than 11,500 vehicles. Second, the start and finish points were recorded for all passings (in one direction) attempted or completed within 500 feet each way from the beginning of the barrier line, at both the Missouri standard and the national standard zones. The third class of data resulted from interviewing approximately 1,000 drivers as they left the test section.

Conclusions

Based on the data collected and analyzed for this report, the following conclusions are offered:

1. Some of the traffic operation characteristics on the two marking systems did not appear to differ greatly. Speed values varied only slightly and drivers not familiar with a particular system displayed only small differences in their speed characteristics at the two zones. However, in an analysis of the more critical conditions for transverse placement, particularly those cases involving vehicles traveling into a no-passing zone in the face of oncoming traffic and those involving the wider vehicles in the commercial class, the findings show that although the differences in placement were small, the advantage was quite consistently with the national standard marking design. Drivers of the vehicles in these categories were significantly farther away laterally from potential head-on collisions and sideswipe accidents.

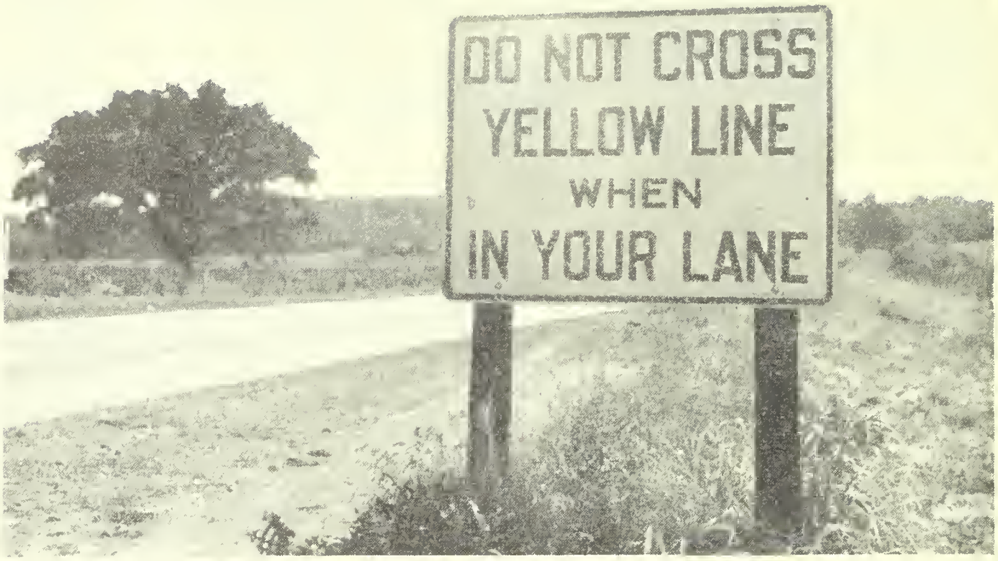

Figure 1.—Standard informational sign used at 3- to 5-mile intervals.

2. In the national standard zone, the average transverse placement of vehicles at night resembled rather closely the daytime condition. In the Missouri zone, the difference between day and night operation was somewhat more marked. It is of interest to note that in each of the six comparative studies on the Missouri zone, the average vehicle was closer to the center line at night than during the day. On the national standard zone, the placement at night for the average vehicle was, at four of the six points studied, farther from the center line than in daylight. At the remaining two, the difference between the day and night placement was less than at the corresponding locations on the Missouri zone.

3. Since observance of no-passing zone markings is very largely a voluntary matter, the extent to which drivers comply is a significant measure of the effectiveness of the markings. The average infringement on the no-passing zone area by drivers observed completing passings was greater at the locations where the barrier line was in the middle of the driving lane, and on this count also, the national standard showed superiority.

4. Direct interview of a representative sample of drivers who had just left the test section revealed no decisive preference for either type of marking. This was as true for Missouri drivers as for drivers from States bordering Missouri and for drivers from other States.

5. Even though some slight improvement in transverse placement occurred when the barrier line was extended forward with a dashed or solid line, driver observance of the extended zone was poor and this cannot be termed an effective means of bettering operating conditions, regardless of the type of no-passing zone.

Location of Test Section

The test section on U S 66 was 16.5 miles in length, extending easterly from Lebanon, Mo., to the Laclede-Pulaski County line. The pavement there was a 20-foot bituminous surfacing over old concrete, flanked by gravel shoulders of adequate width. For the study, approximately half the section length was marked with the national standard no-passing-zone design. The remaining mileage was marked in the Missouri fashion, with the barrier line located in the center of the driving lane.

The terrain traversed by this portion of U S 66 is moderately rolling. No-passing zones are relatively frequent. Traveling westbound through the 16.5-mile section the driver sees 39 no-passing zones; eastbound he traverses 36 zones. The aggregate length of no-passing zone for westbound travel is 6.0 miles; eastbound it is 5.5 miles. These represent restrictions on passing amounting to 36.4 percent and 33.3 percent of the total length traveled by westbound and eastbound drivers, respectively. Signs reading Do Not Cross Yellow Line When in Your Lane (fig. 1) were in place at intervals of 3 to 5 miles along the route, but none was in the vicinity of the zones selected for special study.

To obtain the uniformity desired in the condition of markings throughout the 16.5 miles, the center-line marking was repainted throughout as a dashed, reflectorized white line. Most of the previously existing reflectorized yellow barrier lines in the center of the driving lanes had been painted a year or more earlier and were not too prominent in daylight. These lines were totally obliterated with asphalt paint on the 9.2 miles in the easterly portion of the test section and new reflectorized yellow barrier lines were painted next to the center-line marking in accordance with the national standard. On the 7.3 miles in the western portion, the existing barrier lines were retraced with reflectorized yellow paint.

Two single-direction no-passing zones as nearly identical as could be found were selected for detailed study—one in the eastern, and one in the western portion of the test section. These were designated as sites N (national standard) and M (Missouri standard), respectively. Site N was 4.5 miles west of the Laclede-Pulaski County line near the middle of the length marked with national standard no-passing zones. Figure 2 is a general view of site N. The zone studied is on the grade in the background. The location of site M was 4.4 miles from the east city limits of Lebanon, approximately centered in the test length with the Missouri standard markings. The cover illustration is a photograph of that site, taken from the beginning of the barrier line. The approaches at both sites were on tangent alinement and the sight-distance restriction at the no-passing zone was caused in both cases by summit vertical curves. The standard used in Missouri for this route calls for marking no-passing zones at any point where the sight distance, measured from points 4.5 feet above the road surface, is less than 800 feet. At each of the two study sites, the sight distance from the driver's eye to the pavement surface at the beginning of the barrier line was 550 feet. The zone at site N was marked for westbound traffic and the one at site M for eastbound traffic. At site M the grade in the zone was 4.5 percent; at site N, 4.0 percent.

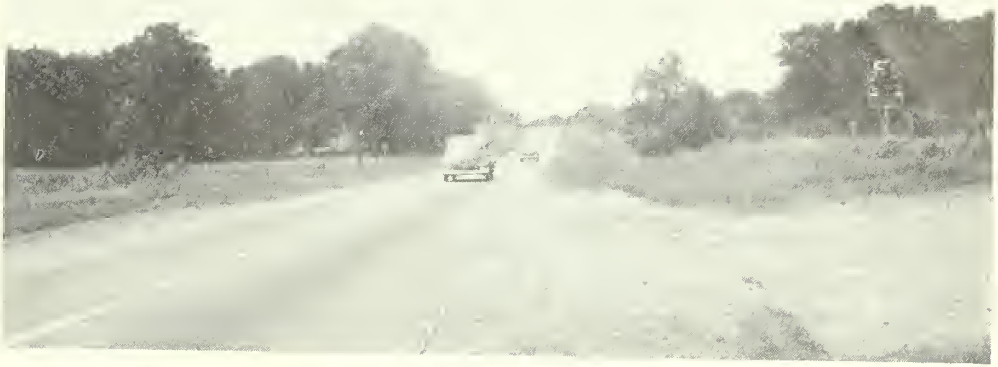

Figure 2.—No passing zone at site N, marked with the national standard, starts near foot of grade in background.

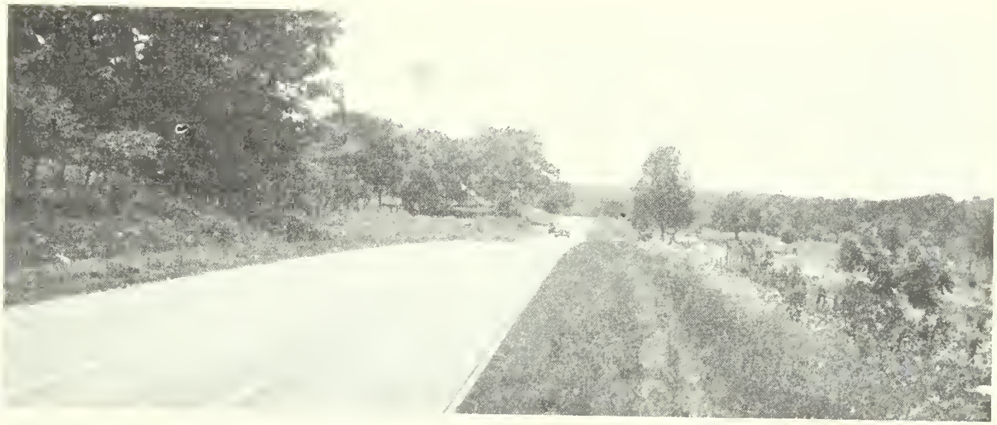

Figure 3.—Two-direction no-passing zone at site B, marked with the Missouri standard in both lanes.

Speed and Placement Studies

Speed and placement data were first recorded for three different study locations in the vicinity of each zone, as indicated below and in figure 4:

No. 1.—500 feet in advance of the start of the barrier line.

No. 2.—At the start of the barrier line.

No. 3.—300 feet beyond the start of the barrier line.

During the course of these studies it was decided to experiment with both a dashed and a solid extension of the yellow barrier line at the approach end. Following study No. 3 a 200-foot dashed extension was painted at both sites. Later the dashed extension of the barrier line was made a solid line. Under these two conditions, three additional locations were studied at sites N and M, as follows:

No. 4.—At the start of the original barrier line (same as No. 2), but with a 200-foot dashed extension of the barrier line.

No. 5.—At the approach end of a 200-foot solid extension of the barrier line.

No. 6.—At the start of the original barrier line (same as Nos. 2 and 4), but with the 200-foot solid extension of the barrier line.

The zones at sites N and M were both single-direction zones. To measure the effect of having a no-passing zone marked for both directions of travel, in which case there is a barrier line at the center of both lanes and a line at the roadway center as well, a site B (for both directions) was also chosen for study. This was located in the section marked with the Missouri standard, 6.1 miles east from the east city limits of Lebanon. The general conditions at site B are portrayed in figure 3.

At each of the three study sites, observations were made in daylight, between 3 and 6 p. m., and at night, between 8 and 11 p. m. Speed, transverse placement, and classification of vehicle types were obtained with the automatic-recording speed-placement equipment. This is the equipment described in the April 1940 issue of Public Roads,[2] except that adding machines were substituted for the graphic recorders to produce a coded record on adding-machine tape. For vehicles entering the no-passing zone, special notation was

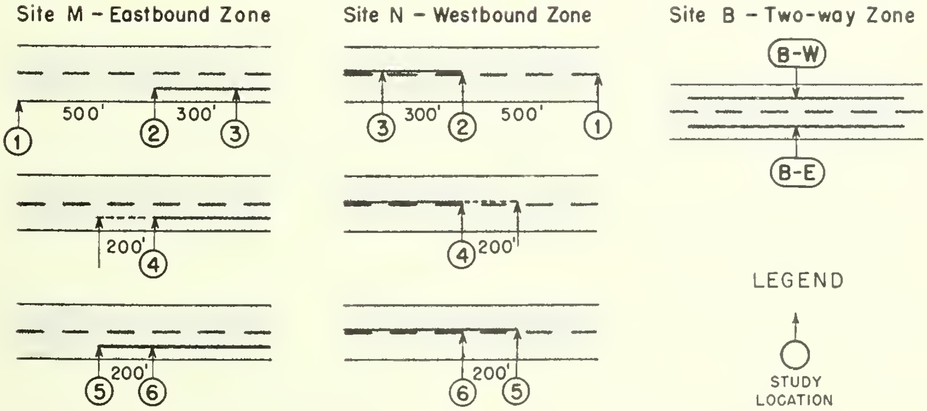

Figure 4.—Positions of study locations at the three no-passing sites: Site M marked with the Missouri standard, site N marked with the national standard, and site B marked with the Missouri standard in a two-way zone.

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[2] *New techniques in traffic behavior studies*, by E. H. Holmes and S. E. Reymer. Public Roads, vol. 21, No. 2, April 1940.

Table 1.—Hourly traffic volumes at study sites

Site and direction of travel	Daylight traffic				Night traffic			
	3–4 p. m.	4–5 p. m.	5–6 p. m.	Average	8–9 p. m.	9–10 p. m.	10–11 p. m.	Average
Site M-1:								
Westbound	115	128	101	115	82	54	45	60
Eastbound	182	186	143	170	64	102	92	86
Total	297	314	244	285	146	156	137	146
Site M-2:								
Westbound	125	132	157	138	67	31	66	54
Eastbound	123	133	102	119	72	72	50	65
Total	218	265	259	257	139	103	116	119
Site M-3:								
Westbound	116	101	109	109	71	55	55	60
Eastbound	110	133	110	118	87	66	50	68
Total	226	234	219	227	158	121	105	128
Site M-4:								
Westbound	125	121	92	113	58	62	73	64
Eastbound	119	103	129	117	75	60	48	61
Total	244	224	221	230	133	122	121	125
Site M-5:								
Westbound	125	117	121	121	92	65	83	80
Eastbound	111	130	128	123	51	79	49	60
Total	236	247	249	244	143	144	132	140
Site M-6:								
Westbound	128	126	147	134	60	61	78	67
Eastbound	117	154	119	130	50	79	37	55
Total	245	280	266	264	110	140	115	122
Site N-1:								
Westbound	132	100	106	113	87	56	69	71
Eastbound	106	103	120	110	69	53	51	57
Total	238	203	226	223	156	109	120	128
Site N-2:								
Westbound	130	131	150	137	123	148	156	142
Eastbound	122	130	115	122	77	97	84	86
Total	252	261	265	259	200	245	240	228
Site N-3:								
Westbound	160	183	170	171	119	80	72	90
Eastbound	126	159	172	152	83	58	68	70
Total	286	342	342	323	202	138	140	160
Site N-4:								
Westbound	120	121	112	118	82	86	84	84
Eastbound	131	135	136	134	52	57	55	55
Total	251	256	248	252	134	143	139	139
Site N-5:								
Westbound	132	96	83	104	95	52	83	77
Eastbound	106	115	77	99	47	57	62	55
Total	238	211	160	203	142	109	145	132
Site N-6:								
Westbound	122	105	97	108	58	73	52	61
Eastbound	120	127	103	117	42	51	62	52
Total	242	232	200	225	100	121	114	113
Site B:								
Westbound	139	200	148	162	91	75	72	79
Eastbound	131	122	99	117	65	60	72	66
Total	270	322	247	279	156	135	144	145

made on the tape if opposing traffic was within 300 feet of the observed vehicle at the time its placement was recorded. In addition, an observer maintained a record of the start and finish of all passings attempted or completed by traffic traveling toward the zone within a distance of 500 feet in each direction from the approach end of the barrier line. A series of posts set at 100-foot intervals, with reflectorized stripes indicating distance from the beginning of the barrier line, facilitated these observations, particularly at night. Hourly traffic volumes by direction were read from traffic counters throughout the period of study.

Table 2.—Traffic classification

Vehicle type	Proportion of total traffic	
	Daylight (3-6 p. m.)	Night (8-11 p. m.)
	Percent	*Percent*
Missouri passenger cars	34.5	34.0
Foreign passenger cars	45.5	23.3
Single-unit trucks[1]	11.1	8.3
Truck combinations	7.8	32.6
Busses	1.1	1.8
Total	100.0	100.0

[1] Includes panels and pick ups.

Many Out-of-State Drivers

The test section carried a daily traffic volume averaging between 3,500 and 4,000 vehicles during the period of the study, June 30 to July 15, 1949. July 4 fell on a Monday, making a long holiday week end, but the traffic flow nevertheless was remarkably uniform throughout the observations. The average traffic for all daylight hours of study on the Missouri standard, site M, was 251 vehicles per hour; at site N it was 247 vehicles per hour. Corresponding volumes during the night studies were 130 vehicles per hour at site M and 150 vehicles per hour at site N. The volumes at site B averaged 279 vehicles per hour in the daylight hours and 145 vehicles per hour at night. This relative stability in the traffic volume factor lends reliability to the comparison of driver practices under the several conditions studied. Table 1 shows the hourly variations and the day and night average volumes at each site.

A detailed classification count made on July 5 is shown in table 2. Of principal interest is the high percentage of foreign (out-of-State) passenger-car traffic, particularly in the daylight hours, and the large proportion of truck combinations at night. The proximity of the holiday undoubtedly accounted in some measure for both conditions, but other observations on this route have consistently shown relatively large percentages of foreign passenger-car and nighttime commercial traffic.

Driving Speeds

The information obtained on traffic speeds indicates also that operating conditions at the two principal test sites were substantially similar. The daytime speed at site M averaged 52.5 miles per hour for vehicles approaching the no-passing zone, and at site N the average speed was 52.3 miles per hour. Within the no-passing zones, vehicles averaged 50.1 miles per hour at site M and 50.4 miles per hour at site N. The slight decrease in speed is accounted for, at least in part, by the grade of approximately 4 percent that vehicles ascended in passing through the zone. Nighttime speeds were slightly lower but followed the same general pattern of decreasing slightly upon entering the no-passing zone.

At site B, where only a single study within the two-direction zone was made, speeds were found to be a little lower than at sites M and N. The daytime average was 48.4 miles per hour for eastbound and 47.6 miles per hour for westbound traffic during daylight hours. Corresponding night speeds were 45.3 and 49.4 miles per hour.

The difference in the effect of the two barrier-line locations on the driving speeds of Missouri and foreign vehicles is of some importance in appraising the no-passing-zone designs. Table 3 shows that foreign passenger-car drivers traveled at a slightly higher average speed than Missouri drivers on the approach to both types of no-passing zones, but tended generally to reduce their speed more than Missouri drivers as they traveled through the zone. The one exception was the case of foreign drivers in the national standard zone at night, where the average speed actually increased 0.9 mile per hour.

Perhaps of most significance is the fact that in every instance the speed decrease occurring as vehicles traveled into the no-passing zone was greater both day and night for Missouri drivers at the national standard zone than it was for the same drivers at the Missouri zone. Similarly, the speed of foreign drivers consistently showed greater change at the Missouri zone than at the national standard zone. The difference between the reaction of Missouri and foreign vehicle operators was

Table 3.—Relative effect of barrier-line location on speed of drivers familiar and unfamiliar with the markings

Zone marking and study location	Daylight speed		Night speed	
	Missouri drivers	Foreign drivers	Missouri drivers	Foreign drivers
Missouri standard:	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Approaching zone	52.8	53.8	49.1	50.2
Within zone	51.6	50.1	48.9	47.8
Difference	−1.2	−3.7	−.2	−2.4
National standard:				
Approaching zone	52.8	52.9	48.5	49.2
Within zone	50.0	49.6	47.2	50.1
Difference	−2.8	−3.3	−1.3	+.9

greater on the Missouri standard than on the national standard. This finding seems logical because Missouri drivers in general would be more familiar with the widely used national standard than other drivers are with the Missouri type of marking.

Table 4.—Average transverse placement (distance from center line of vehicle to center line of roadway) at the several study sites

Study location and barrier-line type	Transverse placement in daylight					Transverse placement at night				
	Missouri passenger cars	Foreign passenger cars	Single-unit trucks	Truck combinations	All vehicles	Missouri passenger cars	Foreign passenger cars	Single-unit trucks	Truck combinations	All vehicles
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Site 1:										
Missouri standard	5.08	5.10	5.33	5.15	5.10	4.61	4.67	4.90	4.90	4.70
National standard	4.16	4.04	4.50	4.60	4.22	3.89	3.82	3.90	4.57	4.09
Site 2:										
Missouri standard	4.34	4.19	4.52	4.92	4.34	4.37	3.90	4.20	4.45	4.21
National standard	4.23	4.07	4.43	4.71	4.21	4.18	4.22	4.56	4.76	4.29
Site 3:										
Missouri standard	4.73	4.61	4.99	4.80	4.71	4.28	4.71	4.20	4.70	4.57
National standard	4.73	4.61	5.29	5.29	4.73	4.60	4.50	4.50	5.04	4.62
Site 4:										
Missouri standard	4.37	4.13	4.68	4.78	4.33	4.20	4.09	4.60	4.57	4.28
National standard	4.16	4.01	4.62	5.03	4.30	4.18	4.04	4.74	4.97	4.45
Site 5:										
Missouri standard	4.29	4.08	4.55	4.71	4.25	3.89	4.08	4.10	4.31	4.10
National standard	4.19	3.69	4.35	4.57	4.03	4.07	3.88	4.40	4.56	4.20
Site 6:										
Missouri standard	4.52	4.29	4.84	4.73	4.48	4.22	4.21	4.60	4.40	4.30
National standard	4.49	4.20	5.05	4.96	4.50	4.57	4.05	4.80	4.86	4.54
Site B (Mo. standard):										
Eastbound	4.70	4.70	4.65	4.90	4.73	4.66	4.68	4.90	4.80	4.68
Westbound	4.42	4.23	4.62	4.60	4.32	4.28	4.27	4.45	4.54	4.28

Transverse Placement

The lateral position of vehicles on the roadway is one of the most sensitive indications of the differences in traffic operation caused by varying the barrier-line location. Extensive study of the placement data has been made to isolate significant driver-behavior characteristics associated with the two types of marking. The dimension used to identify vehicle placement is the distance in feet from the center line of the roadway to the center line of the vehicle.

A compilation of the average placements of vehicles traveling in and through the zones, segregated by day and night and by vehicle type for each study site, is given in table 4. As previously indicated, sites M and B had the barrier line in the center of the driving lane, and site N had the barrier line next to the center-line marking. Site B was the two-direction zone, but site M was marked only for eastbound traffic and site N only for westbound traffic. Since sites M and N were almost identical in physical conditions, traffic volume, and normal speed, the placement of vehicles traveling in the single direction controlled by the respective zones has been selected for analysis.

The values shown in table 4, therefore, pertain to vehicles traveling eastbound at site M, westbound at site N, and for each direction separately at site B. The few vehicles using the left lane for passing in violation of the markings at the several study sites have been omitted from the placement summaries and are treated separately in the section of the report dealing with driver obedience to the no-passing zones.

Close Agreement of Average Placements

The relatively close agreement between the average placement values at locations 2, 3, 4, and 6 on the Missouri and national standard zones is a noteworthy feature of table 4. These four locations were either within the no-passing zone or at the approach end of the normal barrier line, as indicated earlier. Location 5 was at the approach end of the extended barrier line and location 1 was 500 feet in advance of the no-passing zone.

At this latter point, the average lateral position of all vehicles differed 0.88 foot on the two systems in the day study and 0.61 foot at night and, in both cases, the distance from the roadway center line was greater at the Missouri standard zone. The observations at study site M–1 were made July 4 when the weather was partly cloudy, with light showers, and the traffic volume was higher than normal. Site M–1 was also slightly beyond the crest of a vertical curve for eastbound drivers who had just left another no-passing zone about 150 feet before their vehicle placement was recorded. Location N–1, by contrast, was on a nearly flat section and the nearest no-passing zone for westbound drivers in advance of this study site was approximately one-fourth mile to the east. This range in conditions is the suspected cause of the differences between the M–1 and N–1 values.

At site B, the horizontal alinement on the two approaches to the observation point varied slightly. Eastbound traffic entered the zone on tangent alinement, while westbound traffic entered on a curve to the left, which condition is believed to account for westbound average placements being consistently closer to the center line at site B–W than at B–E. No physical or other differences that might influence vehicle paths were evident at any of the other study locations at sites M and N.

As will be seen from table 4, the difference in the average placement values for the two no-passing zone designs is least at study sites M–3 and N–3. In the daytime, the average placement of all vehicles within the Missouri zone was 4.71 feet, measured from center of vehicle to center of roadway, and on the national standard it was 4.73 feet. The corresponding night values were 4.57 and 4.62 feet. These data were recorded at a point 300 feet within each of the test zones, which is certainly one of the more critical locations.

Effect of Traffic Conditions

The data in table 4 include vehicles moving through the test zones singly and in groups, and with and without traffic in the opposing lane. To study these several movement categories independently, the total traffic through the test zones was classified into three groups as follows:

1. Free moving.—Vehicles that were at least 6 seconds behind any other vehicle traveling in the same direction, and were not within 300 feet of any vehicle traveling in the opposite direction.

2. With opposing traffic.—Vehicles that were within 300 feet of a vehicle traveling in the opposite direction.

3. Others.—All other vehicles not in the first two classes, except those that were making a passing maneuver.

Because of the marked similarity in average placements, all passenger-car data from the daytime studies were segregated into these three categories and standard deviations computed for each of the comparative studies. Table 5 shows that, with a single exception (passenger cars with opposing traffic in the national standard zone, study site N–3), the dispersion of placements is greater on the national standard marking than it is on the Missouri marking.

It might be expected that the placement distribution of those vehicles with opposing traffic would always show less dispersion than the distribution for vehicles in the other two movement classifications, because of the imminent danger of colliding head-on with vehicles in the left lane. At all but one of the studies on the national standard this was true, but at three of the six studies on the Missouri standard zone the dispersion was

Table 5.—Standard deviations of transverse placement of Missouri and foreign passenger cars in daylight

Study location and barrier-line type	Standard deviation of transverse placement			
	Free moving	With opposing traffic	Other	All movements
Site 1:				
Missouri standard	0.9063	0.7989	0.8443	0.8511
National standard	1.1385	1.1261	1.6160	1.3260
Site 2:				
Missouri standard	.7611	.7376	.7366	.7521
National standard	1.2147	.8481	1.0524	1.0571
Site 3:				
Missouri standard	.7994	.9290	.6962	.8085
National standard	.8943	.7715	.8777	.8921
Site 4:				
Missouri standard	.7150	.7461	.6997	.7367
National standard	.9442	.7998	1.0899	1.0318
Site 5:				
Missouri standard	.7516	.6782	.7658	.7751
National standard	1.1995	.8727	1.0444	1.0253
Site 6:				
Missouri standard	.9203	.6349	.7815	.7796
National standard	.9834	.9101	.8770	.9128

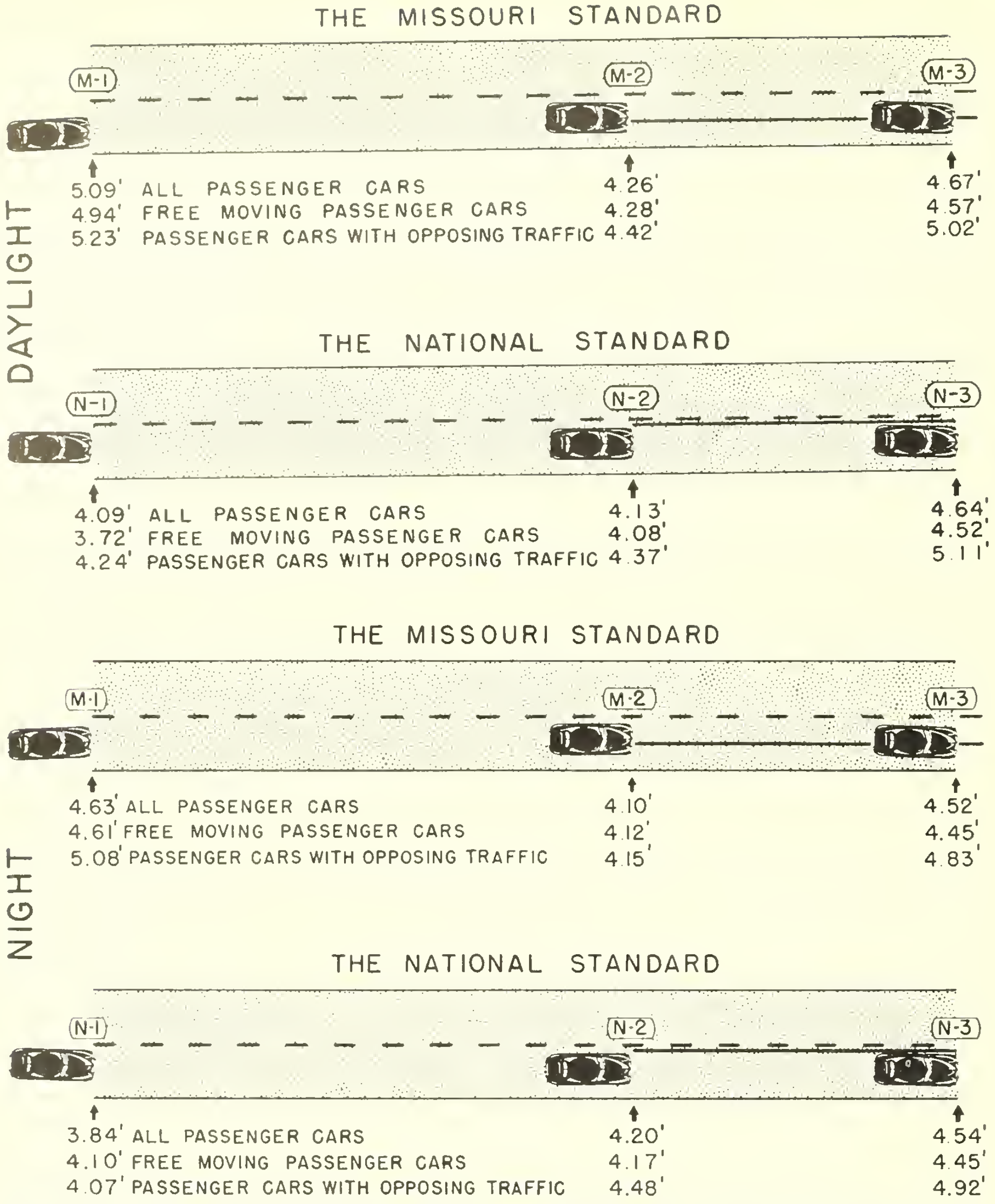

Figure 5.—Transverse placement: Distance from center line of vehicle to center line of roadway for passenger cars in no-passing zones, in daylight and at night.

greater than for one of the other movement classifications. In general, passenger-car drivers using the national standard zone showed somewhat less precision in following a central path than they did when traversing the Missouri standard zone. However, it will be seen that the presence of opposing traffic increased the tendency of drivers on the national standard marking to follow a central path more than it did on the Missouri design. Furthermore, the precision with which the central path was followed on the national standard was slightly superior to that found on the Missouri marking when passenger cars were 300 feet into the zone and faced with traffic in the opposite lane.

Effect of Light Conditions

Figure 5 shows additional comparative data on passenger-car driver practices at the two test zones, under day and night conditions. As previously explained, the placements at site M–1 appear to have been influenced by local factors and probably should not be compared with those for site N–1 for that reason. However, as vehicles reached the beginning of the barrier line and traveled 300 feet into either type of zone, a definite shift to the right occurred in the vehicle path. This shift ranged from about 0.3 foot to 0.75 foot over the 300-foot section. In the daytime, the amount of the shift was greater on the national standard zone; at night, it was greater on the Missouri zone. The net difference between the average placement within the two zones, however, was less than 0.05 foot, either day or night. It will be seen that under all conditions the maximum lateral shift between locations 2 and 3 occurred when oncoming traffic was in the left lane.

Figures 6 and 7 permit more detailed examination of the placement distributions of this important class of traffic at these two points. Passenger cars at the beginning of the national standard barrier line tended to have a more sprawling distribution pattern than at the Missouri standard. This was true both day and night. Within the zones, the distribution of placements was more compact. For both day and night operation within the zone and on either type of marking, about three-fourths of all passenger cars were driven with their centers in the range of 3.6 to 5.5 feet from the roadway center line.

A measure of the relative effectiveness of the two barrier-line locations in inducing vehicle drivers to stay a proper distance from the center line while traversing a no-passing section appears in table 6. Regardless of light conditions, vehicle type, or class of vehicle movement, the proportion of vehicles with placements less than 4.5 feet from the roadway center in the national standard zone was, with several minor exceptions, smaller than in the Missouri type zone. Taken as a whole, the proportion within 3.5 feet of the center line was slightly smaller on the Missouri standard, but in the significant class of vehicles with opposing traffic, the national standard showed marked superiority in the daytime results and was substantially the same as the Missouri standard at night. A lower percentage of commercial vehicles, in every case, was found within 3.5 or 4.5 feet of the center line on the national standard, and their average placement, both day and night in that zone, averaged approximately 0.3 foot farther from the pavement center.

Nighttime is known to be the critical period for many traffic-control devices. Consequently, special attention has been devoted to day and night comparisons at the two barrier-line locations. Data for study of the differences between average placements of all vehicles day and night are presented in table 7. Where the barrier line was located in the center of the driving lane, the average placement at night was consistently nearer the roadway center than the daytime average, as will be seen from the predominance of minus signs in the first column. This is not as generally true where the barrier line was located next to the center line.

Previous studies have indicated that on a typical roadway section removed from a no-passing zone, drivers follow a path closer to the center line at night. That finding is verified by the data obtained in this study at sites M–1 and N–1. From table 7 it is evident that the Missouri standard no-passing-zone marking offers no effective deterrent to this practice. By comparison, the studies on the national standard zone show that the average placement at night closely approximated the daytime position. In some instances, the average night driver was even slightly farther away from the center line than the average daytime driver. At all six study sites compared, the difference between day and night placement averages on the national standard marking was either less than on the Missouri standard or was such as to provide greater clearance from the center line at night than

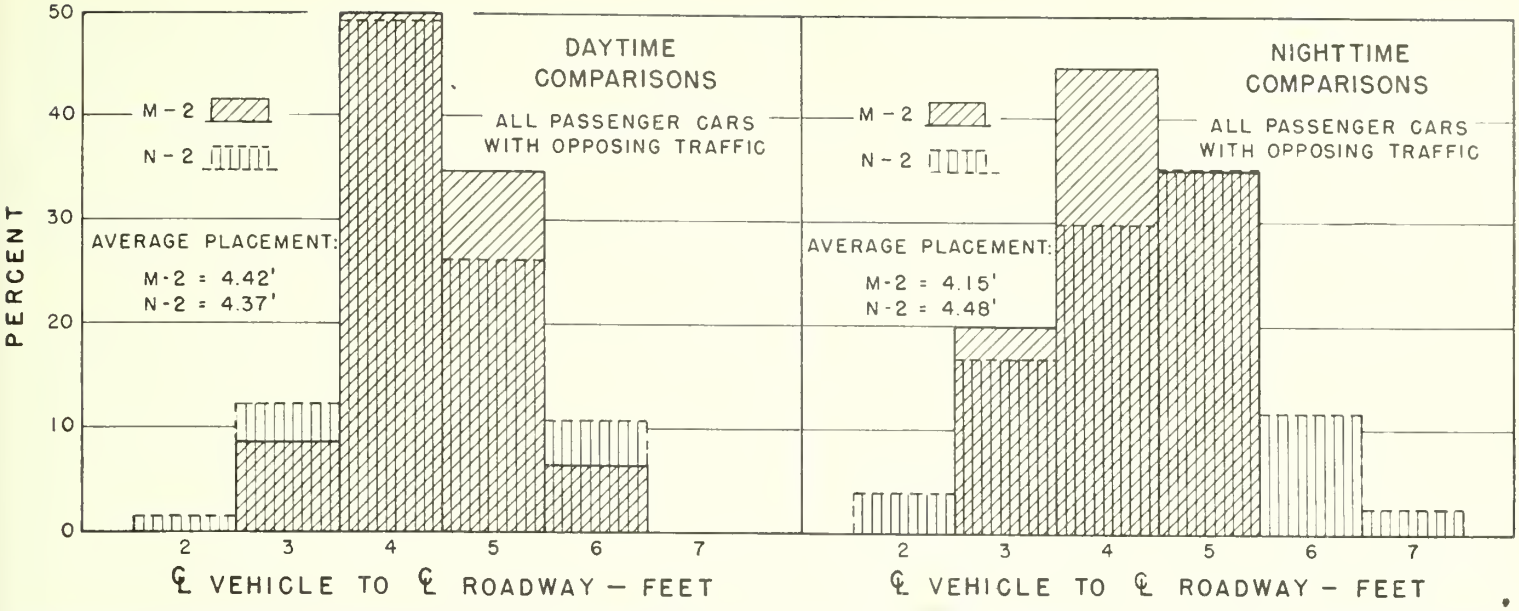

Figure 6.—Distribution of transverse placement of passenger cars at the start of the no-passing barrier line, with opposing traffic, in daylight and at night.

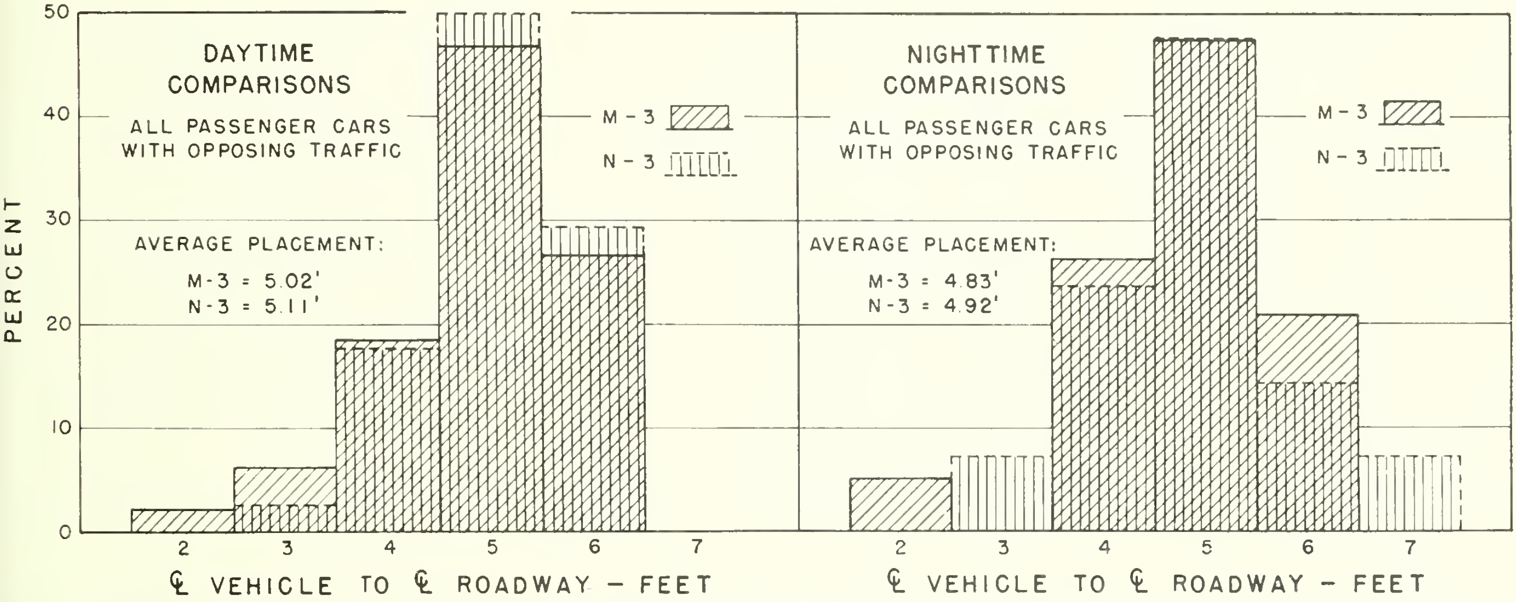

Figure 7.—Distribution of transverse placement of passenger cars 300 feet beyond the start of the no-passing barrier line, with opposing traffic, in daylight and at night.

in the daytime. With the ever-present hazard of sideswipe collisions on narrow pavements, where center-of-the-lane barrier-line marking has been thought to be most applicable, this finding is worthy of attention.

Driver Obedience to Markings

The information recorded on driver compliance with the various types of no-passing-zone markings shows that passings were frequently completed after the barrier line was reached. This may reflect an attitude that the completion of passings is permissible beyond the beginning of the barrier line and not that the maneuver must be completed before reaching it. Whatever the typical driver's reasoning, the average return point of pass completions observed in the vicinity of both the Missouri standard and the national standard zones in this study was beyond the start of the barrier line.

Table 8 is a compilation of the comparative obedience data. The average point of completion was calculated from the beginning of the normal-length barrier line and, in the case of the extended zones, from the start of the extended barrier line.

In general, the completion point was considerably farther into the no-passing zone when the barrier line was located in the center of the driving lane. For the normal zone, the average was 228 feet in the day study and 202 feet at night, as compared with 86 feet and 172 feet, respectively, on the national standard design. No great difference was evident between Missouri and foreign drivers in the daytime results, but at night the latter did not observe either barrier line as well as the local drivers.

On the extended zones, roughly the same pattern of compliance existed. The additional 200 feet of length on the advance end of the barrier lines apparently served only to increase the infringement on the no-passing zones by approximately the same amount.

The data in table 8 indicate that the number of passings recorded was somewhat greater during the studies on the national standard zone. Aside from the effects of alinement and traffic volume which, as previously stated, were quite similar, the sight-

Table 6.—Effect of barrier-line location on average transverse placement, and percentage of vehicles with placement less than 3.5 and 4.5 feet from roadway center while in no-passing zone (sites M–3 and N–3)

Light condition and movement classification	Average transverse placement		Percentage [1] with placement less than—			
			3.5 feet		4.5 feet	
	Missouri standard	National standard	Missouri standard	National standard	Missouri standard	National standard
PASSENGER CARS						
Daylight:	*Feet*	*Feet*	*Percent*	*Percent*	*Percent*	*Percent*
Free moving	4.57	4.52	10.5	13.1	48.7	49.2
With opposing traffic	5.02	5.11	8.2	*2.9	26.6	*20.6
Other	4.57	4.56	5.0	12.2	49.4	*45.4
All movements	4.67	4.64	7.6	10.9	44.2	*42.3
Night:						
Free moving	4.45	4.45	13.8	*11.9	62.1	*56.6
With opposing traffic	4.83	4.92	5.2	7.2	31.6	*30.9
Other	4.43	4.43	11.9	19.1	57.1	*52.4
All movements	4.52	4.54	11.1	13.8	53.4	*49.4
COMMERCIAL VEHICLES						
Daylight:						
Free moving	4.85	5.09	11.8	*3.7	35.3	*11.1
With opposing traffic	4.85	5.71	0	*0	28.6	*0
Other	5.06	5.37	0	*0	18.8	*7.4
All movements	4.92	5.29	4.3	*1.6	27.7	*9.7
Night:						
Free moving	4.51	5.02	7.1	*0	50.0	*9.1
With opposing traffic	4.90	5.02	4.7	*0	33.3	*8.3
Other	4.48	4.81	17.4	*12.0	52.2	*40.0
All movements	4.64	4.91	10.3	*6.3	44.8	*25.0
ALL VEHICLES						
Daylight:						
Free moving	4.62	4.62	10.8	11.4	46.3	*42.9
With opposing traffic	4.98	5.18	6.4	*2.6	27.0	*18.4
Other	4.64	4.65	4.3	10.8	45.2	*41.0
All movements	4.71	4.73	7.0	9.7	41.3	*37.9
Night:						
Free moving	4.46	4.53	11.6	*10.4	58.1	*50.6
With opposing traffic	4.86	4.94	5.0	5.6	32.5	*25.9
Other	4.45	4.52	13.8	17.5	55.3	*49.5
All movements	4.57	4.62	10.7	12.3	49.9	*44.7

[1] Asterisks indicate cases where the percentage for the national standard is lower than (or equal to) the comparable percentage for the Missouri standard.

Table 7.—Difference between average transverse placement of vehicles (all types) in daylight and at night

Study location and movement classification	Difference between placement in daylight and at night [1]	
	On Missouri standard	On national standard
Site 1:	*Feet*	*Feet*
Free moving	−0.34	+0.30
With opposing traffic	−.13	−.06
Other	−.54	−.50
All movements	−.40	−.13
Site 2:		
Free moving	−.16	+.11
With opposing traffic	−.05	+.13
Other	−.16	−.01
All movements	−.13	+.08
Site 3:		
Free moving	−.16	−.09
With opposing traffic	−.12	−.21
Other	−.19	−.13
All movements	−.14	−.11
Site 4:		
Free moving	−.08	+.13
With opposing traffic	−.03	+.23
Other	+.02	+.10
All movements	−.05	+.15
Site 5:		
Free moving	−.17	+.35
With opposing traffic	−.06	+.25
Other	−.35	+.01
All movements	−.15	+.17
Site 6:		
Free moving	−.23	+.14
With opposing traffic	+.18	+.29
Other	−.27	−.13
All movements	−.18	+.04

[1] Minus sign indicates that placement at night was closer to roadway center line than placement in daylight, plus sign indicates placement at night farther away than in daylight.

distance conditions leading to the test sites may have had a relation to the actual frequency of passings recorded. The mile immediately west of site M had 0.3 mile of eastbound no-passing zones while the mile east of site N, by comparison, had 0.45 mile of westbound no-passing zones. This more severe restriction of sight distance available for passing on the mileage approaching site N from the east may account for the somewhat greater number of passings recorded at that location. The difference between the average position of the passing completion points on the two types of barrier-line marking, however, seems more likely to be related to the difference in barrier-line location than to the factor just mentioned.

Obscurement of Barrier Line

When the barrier line is located in the center of the driving lane, it is frequently obscured in part or totally by vehicles on the road ahead. Figure 8 has been constructed to show the comparative conditions of view from the driver's seat when trailing the vehicle ahead at various distances. It assumes that the vehicle ahead is in normal position on a 20-foot pavement and that the approach to the no-passing zone is on tangent alinement. The driver sees the Missouri-type barrier line only by looking under or through the vehicle ahead and, for all practical purposes, his maximum notice of the beginning of the barrier line is the distance between him and the rear of the vehicle ahead. Overtaking and passing studies conducted on rural highways by the Bureau of Public Roads [3] indicate that the average passing vehicle is approximately 55 feet behind the car ahead when it starts into the left lane to pass.

With the barrier line next to the center line, the driver can see to a varying extent around the side of the vehicle ahead and his advance view of the barrier line is considerably longer. If he moves laterally toward the center of the roadway from his normal position, preparatory to passing, his view of the barrier line next to the center line increases rapidly, as indicated by figure 8.

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[3] *Passing practices on rural highways*, by C. W. Prisk. Proceedings of the Highway Research Board, vol. 21, 1941.

Table 8.—Effect of barrier-line location on driver observance of no-passing zones

Light condition and barrier-line type	Passings completed in vicinity of no-passing zones by—					
	Missouri drivers		Foreign drivers		All drivers	
	Number of passings	Average completion point [1]	Number of passings	Average completion point [1]	Number of passings	Average completion point [1]
NORMAL LENGTH ZONES						
Daylight		*Feet*		*Feet*		*Feet*
Missouri standard	20	230	37	227	57	228
National standard	28	66	86	93	114	86
Night						
Missouri standard	14	160	7	286	21	202
National standard	31	87	33	252	64	172
EXTENDED ZONES						
Daylight						
Missouri standard	23	401	46	406	69	405
National standard	33	280	58	342	91	320
Night:						
Missouri standard	11	413	9	528	20	465
National standard	7	307	16	341	23	330

[1] Average completion point is the distance traveled into the no-passing zone before returning to the right lane.

Under other conditions of alinement, the comparative distances at which barrier lines at the two locations could be seen will differ, of course, but there is little doubt that the advantage in driver viewing distance will remain with the national standard in the majority of instances. The lack of advance view is believed to be a prominent factor in the relatively poorer observance found at the Missouri-type zone.

Effect of Extended Barrier Lines

As noted previously, the barrier lines were extended to see if this disadvantage of short advance view of the no-passing zone could be overcome. The 200-foot dashed and solid extensions of the barrier lines did not materially change the average location on the road where passings were completed, but they did influence the average placement of vehicles entering the zones. Table 9 presents the average placements obtained at each zone for three conditions, viz, the normal-length zone, the zone extended at the advance end with a 200-foot dashed line, and the zone extended with a 200-foot solid line. All three studies were made at the beginning of the normal-length barrier line in the two test zones.

The dashed extension on the Missouri standard zone made no appreciable difference in the average day or night placement. On the national standard with the dashed extension, however, the average position of vehicles with opposing traffic was 0.33 foot farther away from the roadway center in the day study and 0.43 foot farther away at night.

With the solid extension, the vehicles with opposing traffic again were affected the most. No great difference was found on the Missouri zone in the daytime, but at night this class of vehicles averaged 0.36 foot farther away from the center line. On the national standard the difference was still greater. The placement results with the solid line extension show that vehicles used a path appreciably farther to the right than they did with no extension or with only a dashed line extension, and that a greater effect generally was caused on the national standard than on the Missouri type zone.

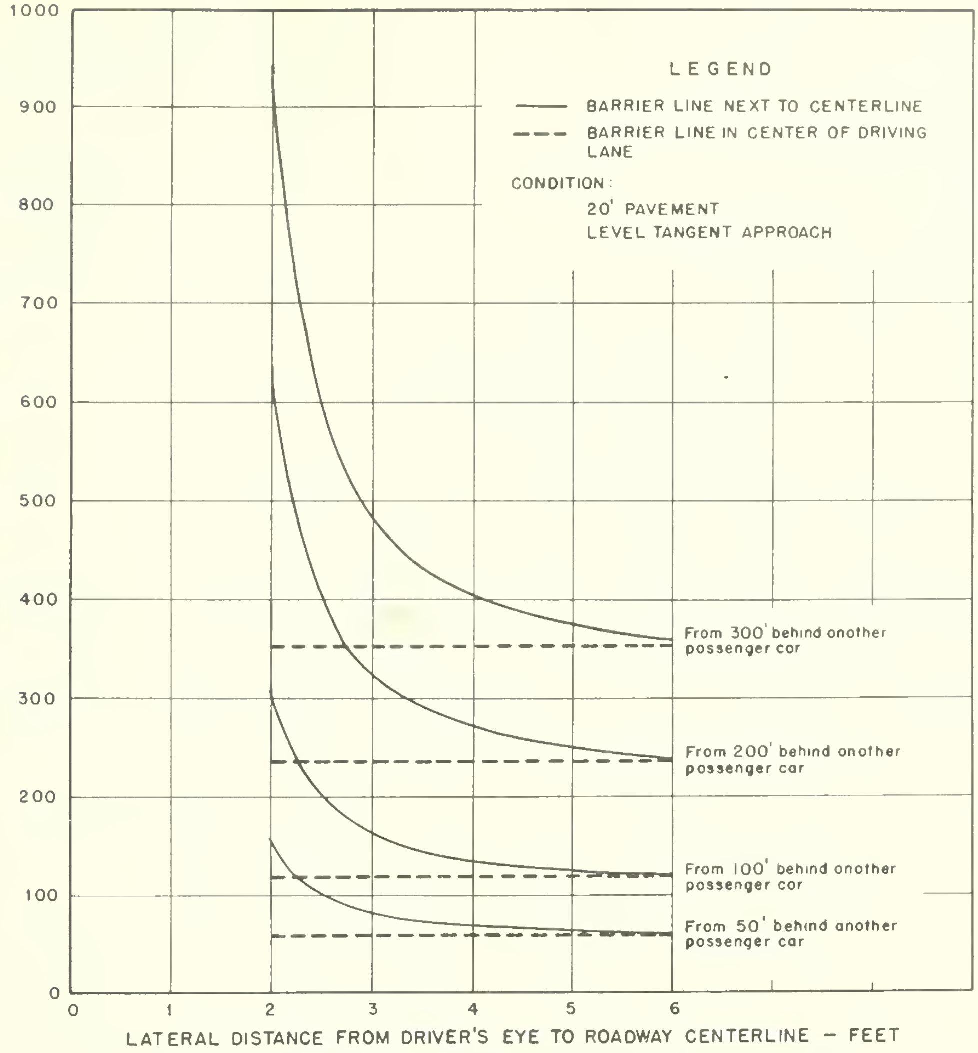

Figure 8.—Obscurement of barrier line by vehicles on the road ahead.

Opinions of Drivers

The Missouri State Highway Department, using the post card questionnaire survey technique, has solicited driver reaction to their no-passing-zone markings on several occasions. One of these surveys made a few years ago, in which almost 12,000 cards were passed out to motorists and 3,063 were returned, indicated a 2½ to 1 preference for locating the barrier line in the center of the driving lane.

As a check on this finding and to eliminate any possibility of bias being caused by having motorists who are intrigued by novel traffic controls responding in greater numbers than those not so interested, a 100-percent sample of driver reaction was obtained as a phase of the study. Interviewers stationed at the extremities of the 16.5-mile test section stopped all outbound traffic and asked drivers these questions:

1. Did you notice the two different systems of marking no-passing zones?
2. Do you have a preference for either system?
3. Which system does your State use?

The State of registration of the vehicle was also noted on the interview form. Station NQ adjoined the national standard, and station MQ the Missouri standard.

A summary of the returns from the 1,005 drivers interviewed appears in table 10. Replies were segregated by drivers of vehicles registered in Missouri, States bordering Missouri, and all other States, to isolate the factor of familiarity. More than 93 percent of all drivers who had traveled over the test section stated that they had noticed the two types of markings. As shown by the reply to the third

Table 9.—Effect on average transverse placement of extending no-passing zone with 200-foot dashed and solid barrier lines

Light condition and movement classification	Placement on Missouri standard					Placement on national standard				
	Normal zone, site M-2	Dashed extension, site M-4		Solid extension, site M-6		Normal zone, site N-2	Dashed extension, site N-4		Solid extension, site N-6	
		Average placement	Change from normal [1]	Average placement	Change from normal [1]		Average placement	Change from normal [1]	Average placement	Change from normal [1]
Daylight movement:	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
All vehicles	4.34	4.33	−0.01	4.48	+0.14	4.21	4.30	+0.09	4.50	+0.29
Free moving	4.36	4.28	−.08	4.43	+.07	4.14	3.92	−.22	4.29	+.15
With opposing traffic	4.53	4.58	+.05	4.66	+.13	4.42	4.75	+.33	4.81	+.39
Night movement:										
All vehicles	4.21	4.28	+.07	4.30	+.09	4.29	4.45	+.16	4.54	+.25
Free moving	4.20	4.20	0	4.20	0	4.28	4.35	+.07	4.43	+.15
With opposing traffic	4.48	4.55	+.07	4.84	+.36	4.55	4.98	+.43	5.10	+.55

[1] Minus sign indicates placement with dashed or solid extension of barrier line was closer to roadway center line than with normal length of barrier line; plus sign indicates placement farther from center line.

Table 10.—Summary of driver interviews at ends of test section [1]

	Drivers from Missouri			Drivers from Illinois, Kansas, Oklahoma, and Arkansas			Drivers from other States			All drivers		
	Station NQ	Station MQ	Total	Station NQ	Station MQ	Total	Station NQ	Station MQ	Total	Station NQ	Station MQ	Total
Different systems noticed:												
Yes	126	135	261	158	101	259	192	227	419	476	463	939
No	5	9	14	10	5	15	17	20	37	32	34	66
Preference:												
Missouri standard	56	68	124	85	62	147	76	132	208	217	262	479
National standard	67	63	130	73	42	115	122	105	227	262	210	472
No choice	8	13	21	10	2	12	11	10	21	29	25	54
System used in own State:												
Missouri standard	---	---	---	2	3	5	2	9	11	---	---	---
National standard	---	---	---	159	100	259	202	225	427	---	---	---
Other	---	---	---	3	0	3	2	3	5	---	---	---
Not known	---	---	---	4	3	7	3	10	13	---	---	---

[1] Eastbound drivers were interviewed at the east end of the test section, at station NQ, adjoining the mileage marked with the national standard; westbound drivers at the west end, at station MQ, adjoining the mileage marked with the Missouri standard.

question, practically every out-of-State driver knew also that his State placed the barrier line next to the center line.

When asked for an expression of preference, Missouri drivers, border-State drivers, and other drivers were all quite evenly divided between the two systems. Neither interview station was within sight of a no-passing-zone marking, but drivers tended to favor the marking standard they had more recently seen. Missouri drivers at station NQ, for example, who had just seen about 9 miles of the national standard marking, showed a 67 to 56 preference for having the barrier line next to the center line, while Missouri drivers at station MQ, having just passed about 7½ miles of the Missouri standard, voted 68 to 63 for the barrier line at the center of the driving lane.

Summary of Findings

The findings of this special study of no-passing-zone marking designs are of necessity based on observation and analysis of traffic performance in a State where only one marking design generally prevails. That one design, which prescribes the position of the barrier line at the center of the driving lane, rather than next to the center line, is the exception rather than the rule among the States. The operating experience from which this report has been prepared may not, therefore, be precisely representative of the performance that would be found if the situation were reversed and the Missouri design were in widespread use. To the extent possible, the effects of familiarity have been explored and discussed so that this factor and the principal data will appear in their proper relation.

The major results derived from comparative study of vehicles traversing the two types of no-passing-zone marking are as follows:

1. Average operating speeds 500 feet in advance of the no-passing zones compared were almost identical, and were slightly over 52 miles per hour for vehicles proceeding toward the zone. At a point 300 feet within each of the zones the general average speed level was lower by 2 to 3 miles per hour, and the greater decreases occurred with foreign drivers on the Missouri type marking and with Missouri drivers on the national standard marking. The difference between the Missouri and the foreign drivers' reaction to the zone, measured in terms of that speed change, was larger at the Missouri zone, probably because Missouri drivers were better acquainted with the conventional barrier-line location than foreign drivers were with the center-of-the-lane position used throughout Missouri.

2. A remarkable similarity exists between the average placement values obtained in comparable studies at the two sites for all locations at or beyond the start of the barrier line. The least difference between average placements at comparable locations in the two zones was observed at a point 300 feet beyond the beginning of the barrier line. Here, on the Missouri marking, the average placement of all vehicles, expressed in terms of the distance from vehicle center to roadway center, was 4.71 feet in the day study and 4.57 feet at night. Corresponding values for the national standard zone were 4.73 feet and 4.62 feet.

At the two-direction no-passing zone marked with the Missouri design, the average placement for the direction with a tangent approach was 4.73 feet in the daytime and 4.68 feet at night. For the opposite direction, the respective values were 4.32 feet and 4.28 feet, indicating only a small difference between average placements on the one-direction and the two-direction zones.

3. The average placement of vehicles at a point 300 feet within both types of no-passing zones was from 0.3 to 0.75 foot farther from the center line than it was for vehicles at the beginning of the barrier line. The maximum differences occurred when opposing traffic was in the left lane. At both types of zones, about three-fourths of all passenger cars traveled with their centers in the range of 3.6 to 5.5 feet of the center line when they were actually within the zone.

4. Detailed analysis of the placement distribution pattern shows interesting differences between the results obtained at the two test zones. On the national standard, vehicle paths were somewhat more widely dispersed than on the Missouri zone. Statistical measures of dispersion for the placement data show that passenger cars, for example, did not generally follow a central path as precisely on the national standard as they did on the Missouri zone. However, it is significant that less dispersion in vehicle placements occurred within the national standard zone than in the Missouri zone when oncoming traffic was in the opposing lane.

5. Regardless of light condition, vehicle type, or class of vehicle movement, the percentage of vehicles with their centers within 4.5 feet of the center line of the pavement was, with a few minor exceptions, less at a point 300 feet within the national standard zone than it was at the corresponding location marked with the Missouri design. The percentage of vehicles that had placements of 3.5 feet or less was a little lower in the Missouri zone when all classes of vehicle movement were considered together. However, a much smaller percentage of the vehicles with opposing traffic on the national standard had placements in this range than on the Missouri standard during the day. At night the corresponding percentages were approximately equal. Without exception, fewer commercial vehicles were centered within 3.5 or 4.5 feet of the roadway center in the national standard zone. Their average placement was also farther from the center line, 5.29 feet on the national standard as compared with 4.92 feet on the Missouri zone in the daytime, and 4.91 feet as compared with 4.64 feet at night.

6. Study of the difference between day and night placement data at all comparable sites reveals that the night averages were generally closer to the daytime averages where the barrier line was next to the center line. At a number of the studies on the national standard marking, the night placements actually averaged slightly farther to the right at night than in the daytime. This did not occur at any of the studies on the Missouri zone.

7. The average driver completing a passing maneuver in the vicinity of the test zones overran the Missouri type no-passing marking by 228 feet in the daytime as compared with 86 feet on the national standard zone. At night the difference was not as great but the national standard zone was still shown more respect. Extension of the barrier line at the advance end with dashed or solid lines resulted in lengthening the average violations roughly by the amount of the extension, but the extended barrier lines seemed to encourage drivers to drive somewhat farther from the center line as they approached the zone. This effect was more pronounced where the barrier line was next to the center line. The driver's advance view of the beginning of a barrier line that is located in the center of the lane is definitely hampered by the geometric limitations involved as vehicles trail one another on the approach to the zone. This factor could easily be responsible for the relatively poorer observance recorded at the Missouri-type zone.

8. A 100-percent sample of drivers interviewed immediately after traveling some dis-

(Continued on page 36)

Age-Strength Relations for Air-Entrained Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by FRANK H. JACKSON
Supervising Highway Physical
Research Engineer

Although it is generally recognized that entrained air in concrete has an adverse effect upon strength, exact information as to the amount of reduction to be anticipated at the later ages has been lacking. Data which are presented in this article indicate that for the usual class A structural concrete (6 sacks per cubic yard and 3 inch slump) a reduction in compressive and flexural strength of approximately 10 percent can be anticipated for ages up to and including 5 years. This is on the basis of approximately equal cement content and slump for both types of concrete, and with the sand content of the air-entrained concrete reduced by an amount approximately equal to the volume of entrained air. The tests also reveal the influence of the chemical composition of cement upon the rate of strength development. They show that the strength of concrete containing type II cement, although comparatively low at early ages, may exceed the strength of concrete containing typical type I cement (high in tricalcium aluminate) at later ages by a substantial amount.

THE tests herein reported were made primarily to obtain data on the rate of strength development of air-entrained concrete for ages up to 5 years as compared with the strength of similar concrete without air entrainment.

As the result of over 10 years practical experience in the use of air-entrained concrete, most engineers have become convinced of the value of air entrainment to improve the durability or weather resistance of concrete. It has been demonstrated both by laboratory research and by experience that the resistance of concrete to frost action can be increased many fold by the introduction of entrained air within the range of 3 to 6 percent. Unfortunately, however, this improvement in durability is almost always accompanied by a reduction in strength which, at the conventional testing periods of 7 and 28 days and for concrete containing approximately 6 sacks of cement per cubic yard, usually averages around 10 percent. This tendency to lower strength has worried many structural designers ever since air entrainment was introduced, not so much because of the early strength loss as the fear that still further reduction in strength might occur with increased age.

In the original program prepared in 1944, it was planned to make freezing and thawing tests as well as strength tests. However, the necessity for using the laboratory's refrigeration equipment for work connected with the war forced the abandonment of the freezing and thawing tests, with the result that only that portion of the program involving the strength tests was carried through to completion.

Conclusions

This report presents a limited amount of data which indicate that the loss in compressive and flexural strength, at least insofar as one class of concrete is concerned, when the specimens are stored continuously moist, remains reasonably constant at about 10 percent up to an age of 5 years. The test results indicate that:

(1) For a typical class A structural concrete (6 sacks per cubic yard and 3-inch slump), the reduction in strength due to air entrainment will average approximately 10 percent for all ages up to 5 years.

(2) Concrete containing cements relatively low in tricalcium aluminate (4 to 7 percent), although lower in strength at 7 and 28 days than similar concrete containing cements relatively high in tricalcium aluminate (11 or 12 percent), will develop substantially higher strengths at ages of 1 year or more.

Materials and Proportions

Eight portland cements, representing four brands, were used in these tests. The chemical and physical properties of the cements are given in table 1. In this table cement brand No. 1 is represented by cements A and B, brand No. 2 by cements C and D, brand No. 3 by cements E and F, and brand No. 4 by cements G and H. Cements A, C, E, and G were submitted as type I and cements B, D, F, and H as type II cements. However, as will be seen from the analyses, only two of the nominally type II cements (B and H) complied with the requirements of type II with respect to the computed tricalcium silicate (C_3S not to exceed 50 percent). Cement D was just over the limit on C_3S with 51 percent, and cement F was further over the limit with 53 percent C_3S. Likewise cement C, although classified as type I, was relatively low in C_3A. It was, however, high in C_3S with 54 percent. Cements A, E, and G were typical type I cements relatively high in C_3A. All in all, the eight cements cover fairly well the range in physical and chemical properties of the types of portland cement most frequently used in highway work.

The values for C_3A as reported in table 1 are lower than those usually reported from these sources. This is partially explained by the method of making the iron oxide and alumina determinations. At the time these analyses were made (in 1944) the Bureau was still using the Jones Reductor for reducing the iron and titrating with potassium permanganate solution. When titanium dioxide is present in a cement it is also reduced by the zinc of the Jones Reductor and is titrated along with the iron. This results in a high value of iron oxide. Since alumina is calculated by subtracting the iron oxide from the total of the ammonium hydroxide group, it is correspondingly low. In the standard procedure of the American Society for Testing Materials, iron oxide is determined by reduction with stannous chloride which does not reduce titanium. However, in the absence of an actual correction for titanium dioxide, the alumina value (as determined by A.S.T.M. procedure) will include this constituent and will therefore be somewhat higher than the true value.

For cement relatively high in C_3A, the difference obtained by the two methods is of little or no significance. However, in calculating the C_3A content the differences are

Table 1.—Properties of the cements

		Brand 1		Brand 2		Brand 3		Brand 4	
		Cement A	Cement B	Cement C	Cement D	Cement E	Cement F	Cement G	Cement H
Physical properties:									
Apparent specific gravity		3.14	3.19	3.14	3.16	3.17	3.16	3.13	3.17
Specific surface	cm.²/gm.	1,610	1,610	1,690	1,580	1,810	1,670	1,690	1,720
Autoclave expansion	percent	0.15	0.04	0.10	0.08	0.06	0.01	0.42	0.02
Normal consistency	percent	26.0	25.0	25.0	24.0	25.0	25.0	26.0	25.0
Time of set:									
Initial	hours	4.0	4.0	5.0	4.0	5.2	5.2	5.0	5.0
Final	hours	7.2	6.5	7.8	6.0	7.5	7.5	8.0	8.0
Tensile strength (1:3 mortar):									
At 3 days	lb. per sq. in.	310	275	240	195	325	275	325	290
At 7 days	lb. per sq. in.	385	360	355	275	370	370	410	370
At 28 days	lb. per sq. in.	465	470	430	380	465	445	465	450
Air entrained [1]	percent	3.5	6.6	5.0	4.4	3.9	4.1	5.0	5.2
Chemical analysis (percent):									
Silicon dioxide		21.30	21.85	21.35	21.57	21.05	22.85	20.40	22.70
Aluminum oxide [2]		6.49	4.42	4.66	4.29	6.44	4.09	6.16	4.66
Ferric oxide [2]		2.96	4.48	3.44	4.56	2.56	3.76	3.44	3.44
Calcium oxide		63.10	62.90	63.50	62.55	63.65	64.65	62.70	64.30
Magnesium oxide		2.64	2.73	2.84	2.97	2.43	1.16	3.00	1.25
Sulfur trioxide		1.81	1.64	1.68	1.51	1.70	1.22	1.88	1.48
Sodium oxide		.27	.23	.21	.23	.19	.17	.27	.20
Potassium oxide		.68	.44	.59	.55	.48	.47	.52	.54
Loss on ignition		1.04	1.36	1.97	2.14	1.50	1.78	1.62	1.42
Chloroform soluble		.006	.004	.006	.007	.003	.005	.006	.008
Computed compound composition (percent):									
Tricalcium silicate		42	50	54	51	48	53	48	48
Dicalcium silicate		30	25	20	24	24	26	22	29
Tricalcium aluminate [2]		12	4	7	4	12	4	11	7
Tetracalcium aluminoferrite		9	14	10	14	8	11	10	10
Calcium sulfate		3	3	3	3	3	2	3	3

[1] A.S.T.M. standard method C 185-44.

[2] Ferric oxide determined by use of Jones Reductor. See text, page 31, third column.

reflected fourfold. This fact, and the practice of rounding compound composition to the nearest whole percentage, means that a difference of only 0.2 to 0.3 percentage points in the value for alumina can mean as high as 2 percentage points difference in calculated C_3A. In the case of low C_3A cements, this may easily affect the classification of the cement as to type.

In order to avoid misunderstanding and to promote uniformity, the Bureau discontinued the use of the Jones Reductor shortly after the analyses reported in this article were made, and the standard A.S.T.M. procedure is now used for all work. Unfortunately, samples of the particular cements used in this study were not available for a repetition of the iron oxide determinations.

Aggregates for the concrete were sand and gravel from the Potomac River. The sand was well graded, with a fineness modulus of 2.62. The gravel was uniformly graded from No. 4 up to 1½-inch size. Both aggregates met the conventional physical test requirements for quality. However, attention should be called to the fact that Potomac River gravel consists essentially of siliceous particles, principally quartz and sandstone. Aggregates of this type frequently produce concrete of relatively low flexural strength as compared with aggregates which are predominately calcareous in nature. This accounts for the relatively low values for modulus of rupture which were obtained in this study.

Test Series

Two series of tests were made, one without the addition of an air-entraining agent and the other in which sufficient neutralized Vinsol resin was added to produce an air content of approximately 4 percent.[1] For each type of concrete the same weight proportions were used with all eight cements with the result that the individual cement contents varied slightly from the nominal value of 6.0 sacks per cubic yard due to variations in the water demand of the various cements. However, in no case did the actual cement content vary more than 0.08 sack per cubic yard from the nominal value. In order to maintain the same yield for the air-entrained concrete, the absolute volume of sand was reduced by an amount approximately equal to the volume of added air, the absolute volume of coarse aggregate remaining the same. The water content was also reduced, as needed, in order to obtain approximately the same slump. Complete mix data, including air contents, are given in table 2.

At the time these tests were started, in 1944, air entrainment was almost universally obtained by the use of a neutralized solution of Vinsol resin, the product of a particular manufacturer. Since that time numerous proprietary air-entraining admixtures have appeared on the market. Many of these are neutralized Vinsol resins marketed under individual trade names. Tests which have been made to date by the Bureau of Public Roads indicate that any admixture which will meet the requirements of the American Society for Testing Materials specification C 260-50T, "Specifications for air-entraining admixtures for concrete—tentative," will be satisfactory for use in the manufacture of air-entrained concrete.

Two types of specimens were made for the tests: standard 6- by 12-inch cylinders for compressive strength tests and 6- by 6- by 20-inch beams for flexural strength tests. On both types of specimens, strength determinations were made after 7 days, 28 days, 1 year, and 5 years moist storage. Values reported are the averages of tests on three specimens, making a total of 384 test specimens for the entire study. Mixing, molding, curing, and testing operations were all conducted strictly in accordance with standard procedures.

Compressive strength data for each cement for both non-air-entrained and air-entrained concrete are given in table 3 with the corresponding data for flexural strength in table 4. The effect of air entrainment on compressive and flexural strength is shown in figure 1 while in figure 2 the same data have been plotted to show the effect of cement composition.

Effect of Air Entrainment

It will be observed that, in the case of all eight cements and for both types of test, the introduction of entrained air caused a reduc-

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[1] Calculated in accordance with the procedure described in the American Society for Testing Materials standard method C 138-44.

Table 2.—Mix data for strength specimens [1]

Cement	Vinsol resin [2]	Cement content [3]	Net water content	Slump	Weight of fresh concrete	Air content [3]
Non-air-entrained concrete:	*Percent*	*Sacks/cu. yd.*	*Gal./sack*	*Inches*	*Lb./cu. ft.*	*Percent*
A		6.0	5.4	2.5	148.6	1.0
B		6.1	5.3	2.7	149.0	.8
C		6.0	5.4	2.3	148.3	1.3
D		6.1	5.3	2.6	148.7	1.2
E		6.0	5.6	2.4	148.3	.8
F		6.0	5.5	3.0	148.7	.8
G		6.0	5.5	2.7	148.4	.8
H		6.0	5.5	2.6	147.7	1.4
Air-entrained concrete:						
A	0.0065	6.1	5.1	2.7	144.6	4.0
B	.0075	6.1	5.0	2.6	144.6	4.1
C	.0090	6.1	5.1	2.7	143.9	4.4
D	.0060	6.1	5.0	2.9	144.5	4.2
E	.0105	6.1	5.2	2.7	144.3	3.9
F	.0080	6.1	5.1	2.9	144.7	3.8
G	.0075	6.1	5.2	2.8	144.7	3.7
H	.0100	6.0	5.0	2.6	143.1	5.0

[1] Proportions by oven-dry weight (in pounds), using No. 4 to 1½-inch gravel: Non-air-entrained concrete, 94:164:355; air-entrained concrete, 94:145:355.

[2] Percentage by weight of cement. Neutralized with sodium hydroxide and added in solution form.

[3] Tests made in accordance with A.S.T.M. standard method C 138-44.

tion in strength at all ages. In compressive strength the percentage reduction varied from a low of 5 (cement G at 7 days, cement E at 1 year, and cements B and E at 5 years) to a high of 19 (cement H at 7 and 28 days). In flexural strength the corresponding range was from a low of 3 (cement G at 28 days) to a high of 21 (cement B at 7 days). In compressive strength, the average percentage reductions for all cements were 10 at 7 days, 11 at 28 days, 12 at 1 year, and 11 at 5 years. In flexural strength, the corresponding reductions were 11 at 7 days, 7 at 28 days, 9 at 1 year, and 8 at 5 years. It will be seen from these data that the reduction of strength in both compression and in flexure averaged approximately 10 percent for periods up to and including 5 years, with tests in flexure tending to show somewhat smaller reductions than those in compression.

Table 3.—Compressive strength tests [1]

Cement	Compressive strength of non-air-entrained concrete				Compressive strength of air-entrained concrete				Reduction in strength due to air entrainment			
	7 days	28 days	1 year	5 years	7 days	28 days	1 year	5 years	7 days	28 days	1 year	5 years
	P.s.i.	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*Percent*	*Percent*	*Percent*	*Percent*
A	4,190	5,510	6,790	7,100	3,760	4,950	6,240	6,390	10	10	8	10
B	3,340	5,010	7,300	7,470	2,990	4,590	6,260	7,090	10	8	14	5
C	3,100	4,850	6,200	7,090	2,890	4,290	5,380	6,230	7	12	13	12
D	2,900	4,500	6,470	7,130	2,480	4,100	5,430	6,130	14	9	16	14
E	3,880	5,890	6,810	7,010	3,650	5,260	6,500	6,630	6	11	5	5
F	3,180	4,890	7,130	7,880	2,970	4,520	6,330	6,740	7	8	11	14
G	3,870	5,630	6,830	6,970	3,670	4,950	5,840	6,340	5	12	14	9
H	3,240	5,150	7,270	7,770	2,630	4,180	6,290	6,470	19	19	13	17
Average	3,460	5,180	6,850	7,300	3,130	4,600	6,030	6,500	10	11	12	11

[1] Each value is the average of three tests of 6- by 12-inch cylinders.

Table 4.—Flexural strength tests [1]

Cement	Modulus of rupture for non-air-entrained concrete				Modulus of rupture for air-entrained concrete				Reduction in strength due to air entrainment			
	7 days	28 days	1 year	5 years	7 days	28 days	1 year	5 years	7 days	28 days	1 year	5 years
	P.s.i.	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*Percent*	*Percent*	*Percent*	*Percent*
A	575	635	630	670	505	605	600	640	12	5	5	4
B	515	645	750	775	405	590	655	700	21	9	13	10
C	485	565	660	730	450	545	570	675	7	4	14	8
D	460	595	635	785	385	540	595	715	16	9	6	9
E	545	620	670	690	495	580	590	630	9	6	12	9
F	485	615	700	720	450	560	645	670	7	9	8	7
G	515	610	625	665	490	590	595	620	5	3	5	7
H	470	595	685	735	425	535	640	680	10	10	7	7
Average	505	610	670	720	450	570	610	665	11	7	9	8

[1] Each value is the average of three tests of 6- by 6- by 20-inch beams.

These data indicate that under certain conditions a strength reduction due to air entrainment as high as 20 percent might be obtained without exceeding an air content of 5 percent. At first sight this might seem to be quite serious. However, if we examine the strength values themselves we note that even in the case of cement H, which, with 5 percent air in the concrete, showed the largest reduction in compressive strength, the strength of the air-entrained concrete at 28 days was 4,180 pounds per square inch, which is roughly 40 percent higher than the 3,000 usually assumed for design purposes in the case of class A structural concrete. Thus it will be seen that even on the basis of a 19-percent reduction in compressive strength—the largest reduction in compressive strength found in these tests—there still remains a fairly high factor of safety in the use of air-entrained concrete in cases where compressive strength is used as the basis of design.

In flexure the lowest strength obtained with any cement at 28 days was 535 pounds per square inch. This was air-entrained concrete containing cement H with 5 percent air. As previously stated, the Potomac River gravel which was used in these tests, due to its predominately siliceous composition, produces concrete having low flexural strength as compared with concretes containing other types of aggregates, particularly those which are predominately calcareous in composition. Under these conditions it is difficult to meet the usual strength design requirement of 550 pounds per square inch modulus of rupture at 14 days without using a cement content substantially in excess of the 6.0 sacks per cubic yard used in these tests. It is recommended, in all cases where aggregate of this type is to be used in pavement concrete, that the required cement content be determined by laboratory test. This applies to both non-air-entrained and air-entrained concrete but is particularly important in the case of the latter due to the lower strength which is usually obtained.

Even in cases where the aggregates are of such a nature as to develop high flexural strength (say 700 to 750 pounds per square inch at 14 days) it is good engineering to design the mix by laboratory test in order to realize the saving in cement which the use of high-strength materials may make possible. The cement content which will develop the required flexural strength should be used as the basis of mix design and this can only be determined by test.

Significance of Strength Reductions

It should be noted that the reductions in strength herein reported are based on laboratory tests of carefully controlled concrete mixtures and that they therefore do not take into consideration the improvement in uniformity of field mixtures of air-entrained concrete which may reasonably be anticipated. It is well known that air-entrained concrete is much more workable and therefore can be placed with much less danger of segregation, water gain, etc., than concrete which does not contain entrained air. This is particularly true where the concrete is to be placed in heavily reinforced structural members where workability is of prime importance. Under these conditions, especially, it is felt that the improvement in uniformity of the concrete in the member as a whole will more than compensate for the theoretical loss in strength due to the entrained air as indicated by tests of 6- by 12-inch cylinders. These benefits are hard to measure quantitatively, but they are none the less real as will be attested by construction men generally who have used air-entrained concrete in their work.

There is another point which should be emphasized when studying the significance of strength losses due to entrained air. This is the relatively rapid deterioration of non-air-entrained concrete exposed to severe weathering as compared to concrete containing entrained air. The loss in strength which invariably accompanies deterioration due to the effect of alternate freezing and thawing may very quickly neutralize any initial strength advantage the non-air-entrained concrete may have enjoyed.

Effect of Cement Composition

The rather wide variations in chemical composition of the eight cements used in this study make it possible to study to a limited degree the age-strength relations as they are affected by variations in the three major compounds in cement—tricalcium silicate (C_3S), dicalcium silicate (C_2S), and tricalcium aluminate (C_3A). As previously stated, it was the original intent to obtain a typical type I and a typical type II cement from each of the four companies supplying cement for this investigation. In figure 2 the concrete strengths obtained with the four pairs of cements that were actually supplied have been plotted in such a way as to reveal the effect of variations in composition of each pair separately. The computed percentage of C_3A, C_3S, and C_2S for each cement is also shown at the top of figure 2.

The effect of composition on rate of strength development is immediately apparent. For example, concrete containing cement B with 4 percent C_3A, although with one exception lower in both compressive and flexural strength than concrete containing cement A at 7 and 28 days, is considerably higher in strength at the later ages of 1 and 5 years. The same is generally true of concrete containing cements E and F and cements G and

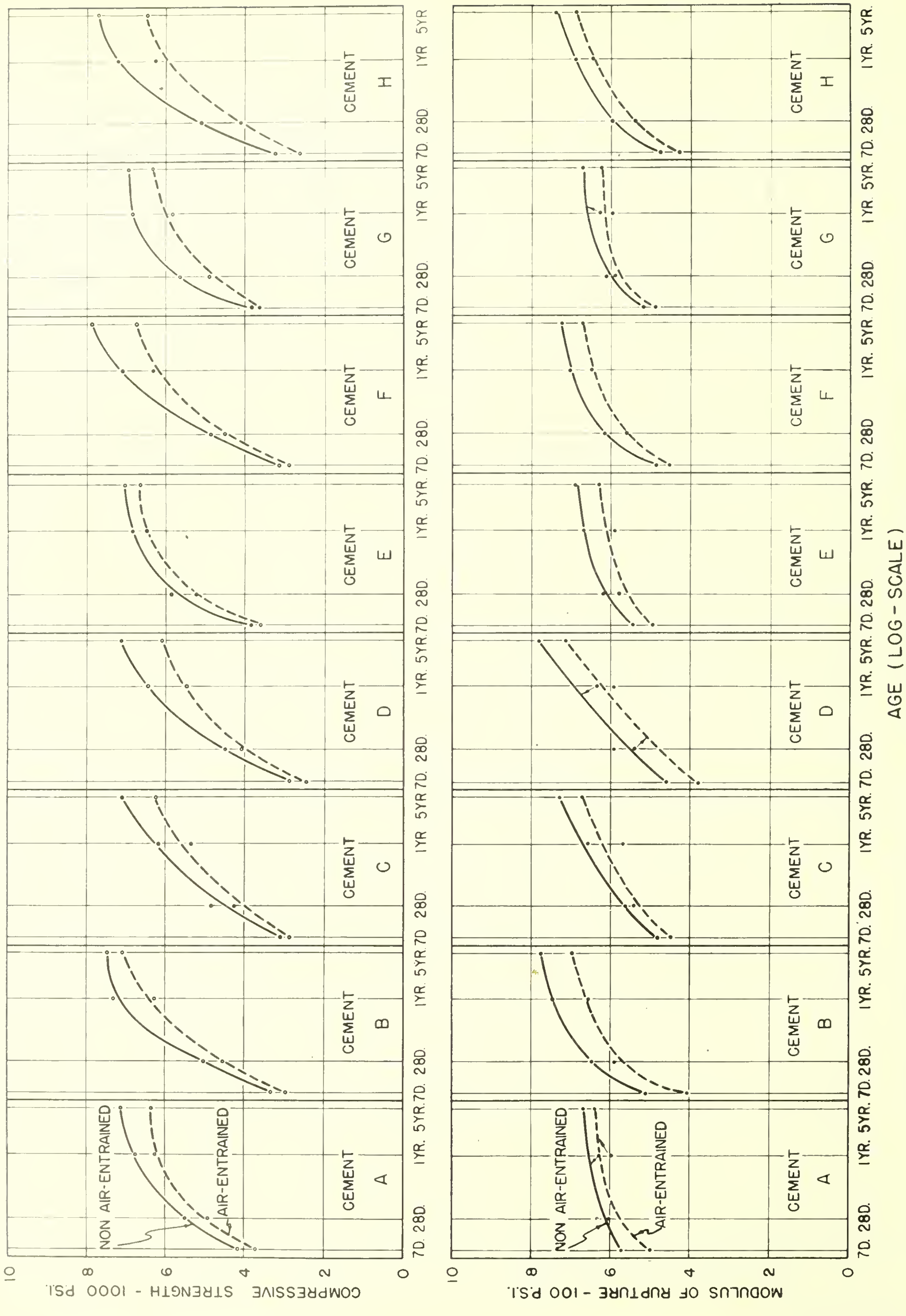

Figure 1.—Effect of air entrainment on the compressive strength (upper chart) and flexural strength (lower chart) of concrete.

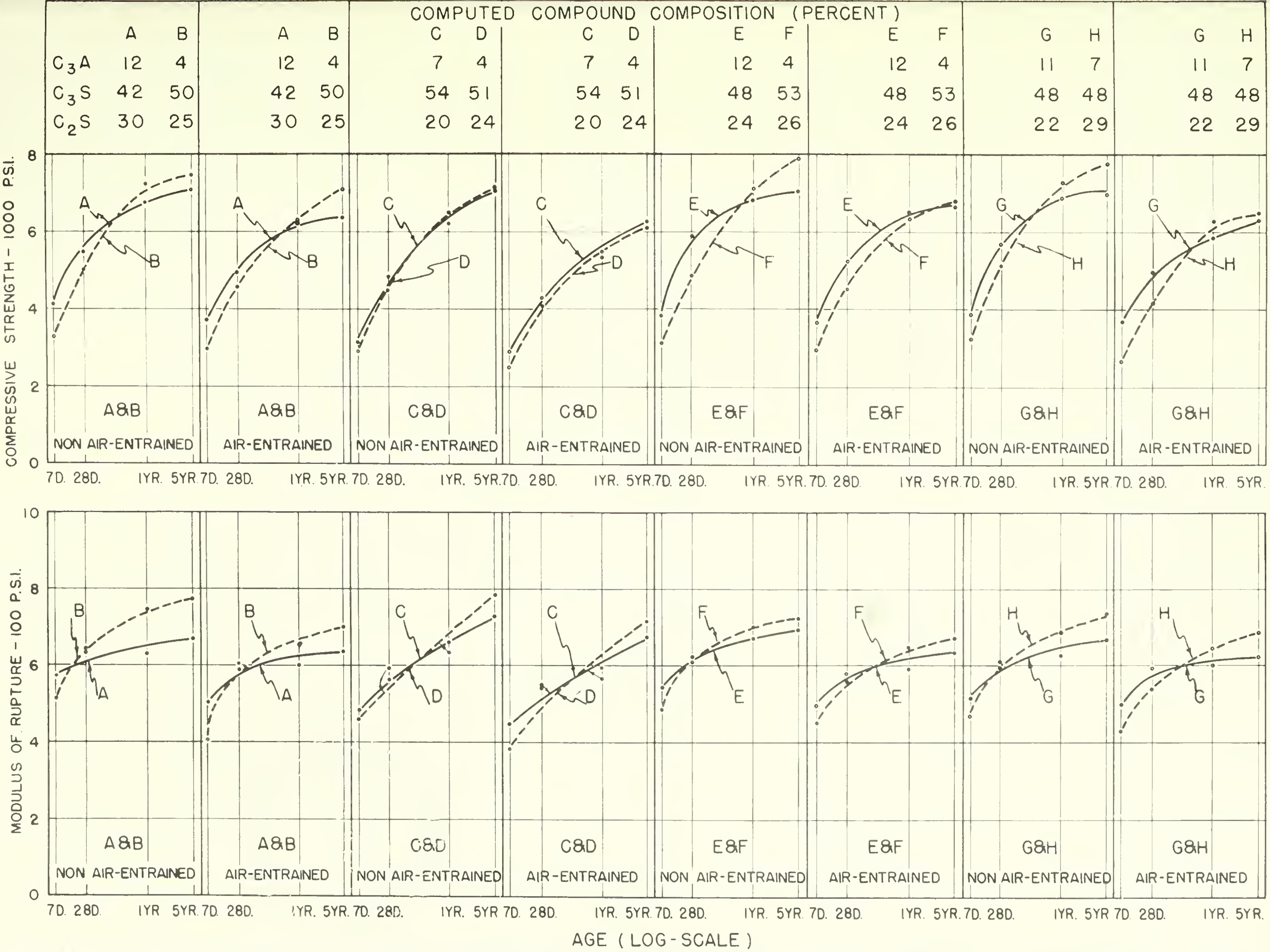

Figure 2.—Effect of cement composition on the compressive strength (upper chart) and flexural strength (lower chart) of concrete. The computed percentages of tricalcium aluminate, tricalcium silicate, and dicalcium silicate in each of the cements are shown at the top of the upper chart.

H where similar variations in C_3A are indicated. Only in the case of cements C and D are the rates of strength development about the same. In this case, both cements are sufficiently low in C_3A to be classified as type II cements although neither would qualify insofar as C_3S is concerned. It may also be noted that the two cements giving the highest compressive strengths at 5 years (F and H) in non-air-entrained concrete are also highest in combined C_3S and C_2S (79 and 77 percent, respectively) while cement G, which shows the lowest combined C_3S and C_2S (70 percent), developed the lowest compressive strength in non-air-entrained concrete at 5 years. In the case of flexural strength, cements B and D (with 4 percent C_3A and 75 percent combined C_3S and C_2S) developed the highest strength at 5 years with cement G having the lowest strength.

The above trends with respect to cemen composition, although not startling, are o interest largely because they verify by actua test the generally accepted theories regardin the effect of cement composition on rate o strength development. They show in genera that concrete containing type II cement, al though low in early strength, is apt to exceed the strength of concrete containing type cement by substantial amounts at later ages

The Effect of Barrier-Line Location at No-Passing Zones

(Continued from page 30)

tance over both types of marking indicated that their preference was far from being one-sided. Of the 1,005 drivers questioned, 479 preferred the Missouri design, 472 preferred the national standard, and 54 had no choice. The expression of Missouri drivers alone was 130 to 124 for placing the barrier line next to the center line, while 21 stated they had no preference. The survey revealed too that over 93 percent of the drivers had noticed the two types of marking, and that most foreign driv ers were aware that their State placed the barrier line adjacent to the center line, whicl is the location recommended as the nationa standard.

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Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.
Title-Sheets for PUBLIC ROADS, volumes 24, 25, and 26.

U. S. GOVERNMENT PRINTING OFFICE : 1952—998627

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1952

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$9,843	$30,684	$15,394	477.3	$8,210	$4,571	123.9	$22,335	$11,261	439.1	$61,229	$31,226	1,040.3
Arizona	804	5,747	4,146	133.0	2,210	1,583	36.8	8,881	6,171	140.7	16,838	11,900	310.5
Arkansas	7,722	6,824	3,760	246.5	4.025	2,011	153.7	16,168	8,233	450.0	27,017	14,004	850.2
California	4,258	38,400	13,759	179.9	18,985	8,366	78.8	78,258	37,784	236.7	135,643	59,909	495.4
Colorado	6,225	4,808	2,271	114.4	3,507	1,946	83.3	11,601	6,299	255.5	19,916	10,516	453.2
Connecticut	6,477	3,500	1,800	13.2	1,709	822	2.0	10,516	5,571	19.2	15,725	8,193	34.4
Delaware	2,610	1,600	800	8.0	2,767	1,032	12.1	3,281	1,630	37.7	7,648	3,462	57.8
Florida	4,964	15,034	7,782	319.8	10,922	5,527	183.9	13,974	6,766	290.6	39,930	20,075	794.3
Georgia	8,156	17,286	8,784	501.0	9,324	4,356	134.0	33,816	16,612	509.1	60,426	29,752	1,144.1
Idaho	5,630	11,600	7,305	283.0	1,221	766	39.5	5,395	3,497	127.3	18,216	11,568	449.8
Illinois	14,120	51,635	28,437	427.0	24,986	12,501	263.7	73,735	37,257	591.4	150,356	78,195	1,282.1
Indiana	12,106	36,119	18,773	206.0	11,441	6,142	226.5	24,316	12,637	252.4	71,876	37,552	684.9
Iowa	2,433	17,712	9,355	583.9	11,111	5,580	507.2	16,870	8,398	687.3	45,693	23,333	1,778.4
Kansas	8,856	11,270	5,482	1,288.4	8,187	4,078	371.8	16,744	8,750	629.2	36,201	18,310	2,289.4
Kentucky	5,213	19,497	10,200	283.4	7,388	3,990	133.6	13,587	6,868	248.8	40,472	21,058	665.8
Louisiana	3,142	18,047	8,676	127.6	13,256	6,136	79.5	17,115	8,553	172.8	48,418	23,365	379.9
Maine	3,039	7,973	4,360	45.7	2,019	1,011	10.1	7,168	3,620	63.0	17,160	8,991	118.8
Maryland	8,230	4,079	1,953	56.1	544	275	11.1	11,163	5,833	24.9	15,786	8,061	92.1
Massachusetts	7,362	7,693	3,849	11.1	2,073	836		50,159	24,360	46.6	59,925	29,045	57.7
Michigan	5,915	32,076	15,972	524.2	9,854	4,975	181.8	56,246	24,731	324.1	98,176	45,678	1,030.1
Minnesota	5,468	11,569	6,086	1,167.3	14,711	7,458	811.2	20,056	11,089	664.7	46,336	24,633	2,643.2
Mississippi	4,702	17,744	9,124	626.0	7,010	3,476	215.1	17,866	9,310	466.5	42,620	21,910	1,307.6
Missouri	11,596	30,615	15,451	890.7	12,226	6,113	301.6	43,362	22,760	648.1	86,203	44,324	1,840.4
Montana	9,919	10,234	6,163	332.0	3,954	2,273	104.3	18,844	11,310	300.0	33,032	19,746	736.3
Nebraska	12,183	12,017	6,294	527.1	8,337	4,160	182.8	17,144	8,510	557.6	37,498	18,964	1,267.5
Nevada	2,494	7,727	6,157	314.3	1,278	1,069	46.9	4,498	3,739	162.5	13,503	10,965	523.7
New Hampshire	2,612	3,661	1,974	24.1	629	314	2.2	6,461	3,219	45.0	10,751	5,507	71.3
New Jersey	2,909	13,212	6,486	34.5	5,354	2,665	9.3	31,948	15,634	21.8	50,514	24,785	65.6
New Mexico	2,130	6,776	4,339	262.9	2,469	1,584	60.7	13,399	8,570	303.8	22,644	14,493	627.4
New York	44,554	78,229	40,881	145.2	13,512	6,262	63.3	103,551	47,271	368.2	195,292	94,414	576.7
North Carolina	6,902	22,815	11,110	418.7	6,007	2,631	169.5	22,658	11,363	405.0	51,480	25,104	993.2
North Dakota	2,394	13,249	6,785	1,429.5	5,554	2,789	510.8	8,713	4,397	762.5	27,516	13,971	2,702.8
Ohio	14,906	36,531	17,944	157.0	13,139	6,485	73.6	74,269	37,777	251.6	123,939	62,206	482.2
Oklahoma	4,448	16,029	8,872	257.3	8,088	4,217	167.5	19,813	10,552	201.6	43,930	23,641	626.4
Oregon	1,836	3,789	2,240	61.4	3,629	2,141	73.3	16,849	9,567	225.2	24,267	13,948	359.9
Pennsylvania	5,098	54,807	27,396	161.7	10,127	5,051	15.6	73,765	36,625	192.8	138,699	69,072	370.1
Rhode Island	1,283	5,850	2,925	45.6	581	290	1.2	18,340	9,104	23.3	24,771	12,319	70.1
South Carolina	3,884	11,487	6,204	381.3	3,610	1,775	156.6	14,865	7,600	342.4	29,962	15,579	880.3
South Dakota	1,737	9,330	5,353	743.8	5,081	3,028	317.3	12,440	7,227	681.1	26,851	15,608	1,742.2
Tennessee	6,294	12,677	6,023	436.9	13,189	6,273	227.8	27,089	12,938	356.3	52,955	25,234	1,021.0
Texas	17,782	11,412	6,230	227.4	12,370	6,349	469.5	57,175	29,538	954.8	80,957	42,117	1,651.7
Utah	2,780	3,648	2,801	70.6	3,418	2,672	65.3	6,386	4,699	93.4	13,452	10,172	229.3
Vermont	1,483	4,600	2,484	48.8	1,699	884	21.2	5,497	2,734	35.9	11,796	6,102	105.9
Virginia	7,946	19,985	9,972	377.7	6,227	3,102	139.5	24,285	12,140	387.6	50,497	25,214	904.8
Washington	2,672	15,423	7,500	265.0	2,770	1,439	58.1	22,470	11,404	133.5	40,663	20,343	456.6
West Virginia	6,914	9,125	4,633	82.5	2,798	1,404	42.4	12,308	6,132	110.7	24,231	12,169	235.6
Wisconsin	5,145	28,370	14,883	593.5	17,255	8,290	215.1	18,927	9,693	407.8	64,552	32,866	1,216.4
Wyoming	1,979	2,072	1,360	22.4	1,856	1,311	46.7	8,051	5,208	162.0	11,979	7,879	231.1
Hawaii	1,782	7,270	3,482	13.0	439	220	5.2	10,648	4,267	24.0	18,357	7,969	42.2
District of Columbia	7,801	84	42	.2				3,252	1,819	.9	3,336	1,861	1.1
Puerto Rico	5,583	8,126	3,848	50.5	3,988	1,940	9.2	8,829	4,021	31.5	20,943	9,809	91.2
TOTAL	336,321	830,047	427,900	15,998.4	345,044	174,169	7,216.1	1,235,086	625,048	14,868.5	2,410,177	1,227,117	38,083.0

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VOL. 27, NO. 3

AUGUST 1952

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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U. S. DEPARTMENT
OF COMMERCE,
WASHINGTON

In this issue: An article on highway transportation economics
(U S 1 near Beltsville, Md.)

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 27, No. 3 *August 1952*

Published Bimonthly

IN THIS ISSUE

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The printing of this publication has been approved by the Director of the Bureau of the Budget, January 5, 1952.

BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

Highway Transportation Economics

y RICHARD M. ZETTEL
ssociate Research Economist,
nstitute of Transportation
and Traffic Engineering,
University of California

PUBLIC ROADS takes pleasure in presenting this stimulating discussion of highway transportation economics by Richard Zettel of the Institute of Transportation and Traffic Engineering, University of California. It is certain to be of keen interest to layman, engineer, and economist alike.

It would require a weighty volume to detail exhaustively the maze of factors that complicate the problems of transportation economics. Many of the factors, as the author points out, are unmeasurable at present and may remain so. It is in the presentation of the problems of highway transportation economics that this article has its strength. Specific solutions are not offered. Indeed, there are no acceptable solutions to many of the problems that exist—at least, none to which all interested parties can now agree.

One of the basic difficulties, Mr. Zettel notes, is to get and retain perspective. What is the proper position of highway transportation in the total transportation field? How should highway costs be assigned among the various classes of motor vehicles, and what responsibility should be assumed by the general public as distinct from highway users? What are our highway needs today, and how shall costs and benefits be related? And what is the public willing to pay? How should highway needs be measured in relation to other needs for the general welfare?

Extensive research may eventually produce enough statistical data to permit derivation of theoretical solutions for some of these problems. But in the final analysis, as the author indicates, many decisions must rest upon collective judgment expressed through legislative bodies. It is hoped, of course, that research will whittle away at the edges of the imponderables and provide bases for sounder judgments which will reduce the imbalances and inefficiencies that may be present in our transportation system today.

As Mr. Zettel points out, we may take comfort from the fact that, despite all the difficulties and confusion, we have a complex yet effective and competent transportation system in this country. The effectiveness of the great productive machine that has made possible our high standards of living cannot be attributed alone to the wealth of our natural resources, the skill of our labor forces, and our efficient use of capital. It rests also, in no small measure, upon this remarkably capable transportation system which we have developed.

And highway transportation, all things considered, is perhaps the most important of all, the author believes. Not only is it a potent competitive force, but it complements all other modes of transportation and plays an indispensable role in the direct satisfaction of human wants. A continuation of the phenomenal progress of the past 50 years is one of the great challenges of the future.

Engineers, economists, and public-finance experts may not all see eye to eye with some of Mr. Zettel's theses and interpretations of available statistics, and publication of the article here does not signify that the Bureau of Public Roads necessarily vouches for it in entirety. The subject is so broad, so little explored, and so unsupported by facts, that lack of agreement is to be expected. Nevertheless, the reading of so thought-provoking a discussion cannot fail to stimulate all who have an interest in highways and transportation.

N discussing a subject as broad as highway transportation economics it is necessary to hoose the issues which seem, to the writer at ast, to be most challenging. What will be ttempted is a portrayal in broad outlines of he place of transportation in general, and ighway transportation in particular, in the odern economy. Also to be pointed up are ome of the broad economic issues which are f great concern to the public generally and hould be of even greater concern to those irectly connected with the transportation ndustry.

BASIC ECONOMIC CONCEPTS

Economics is the study of the utilization of mited resources to satisfy human wants; it s a study of the production, distribution, and onsumption of goods and services. It is asically concerned with the allocation of reources among the various uses to which they may be put. As we shall see, in the modern conomy a considerable share of our economic fforts, of our labor, capital, and managerial bility, is devoted to supplying transportation ervice; and a considerable share of our inomes goes to pay for that service.

Transportation Fundamentals

Transportation service may satisfy human wants directly, as in the case of pleasure driving or vacation travel, or it may play a vital role in the productive process, as in the ase of bringing raw materials to the factory and distributing finished goods to the market. n economic parlance, transportation adds place utility to goods and, hence, increases heir value; otherwise it would not exist. These ideas may seem self-evident; but the ignificance of transportation's role in a progressive economy is not often fully appreciated. In fact, transportation is sometimes ooked upon as a necessary evil, an unfortunate economic waste. Often neglected is the way in which an efficient transportation system increases our standards of living and contributes to our social well-being.

Consider for a moment the primitive American frontier town with its poor transportation acilities, and contrast it with the United States of today and its complex network of ransportation. Only a moment's reflection hows that adequate transportation makes or a tremendous difference in the economy. Of course, transportation has always been vital, even in the primitive economy—even when it was pack-a-back. When early man learned to transport fish and game he improved his standard of living. Transportation meant economic progress. Until iron ore and fuel, not 10 miles apart, could be brought together steel could not be made.

Even today, one of the basic requirements for economic progress in the underdeveloped areas of the world is improved transportation, for the richest of resources have no value unless they can be moved, put into usable form, and then brought to the consumer. In a very real sense, the story of transportation is the story of civilization and economic progress. It is not surprising, then, that some have said the greatest invention of all time was the wheel. The anthropologist and the historian, the political scientist and the

A considerable share of our economic effort is devoted to supplying transportation services. (Washington, D. C.)

sociologist, frequently find explanations of institutional settings and cultural activities rooted in the prevailing transportation patterns of the times.

Transportation and Trade

Transportation is the prerequisite of trade. The travels of a Marco Polo, the explorations of a Columbus, were undertaken for purposes of trade, which presupposes transportation. Without improved transportation a community, or a nation, is limited to the goods it produces for itself. And transportation plays a more subtle role in our economy than merely permitting us to trade goods we have for those we have not.

Much of our modern economic progress may be attributed to regional specialization or division of labor, which tremendously increases our productive capacity by increasing efficiency and making large-scale manufacturing possible—both by bringing together the materials needed and by distributing the products to expanded markets. Consider the steel industry of Pennsylvania, citrus production of California and Florida, cotton and tobacco raising in the South, lumbering in the Northwest, the automobile industry of Detroit—and reflect on whether any of these would have been possible without improved transportation.

Transportation and Prices

Transportation serves a useful function in equalizing and stabilizing prices. Improved transportation effectively stays the development of local monopolies. It assures that a crop failure in one area will not bring disaster, for foods from other areas can be imported. Improved transportation increases competition, for when prices in one area are raised above the cost of production in another area plus the cost of transportation, products will be imported. An important feature of our economic system, directly related to the adequacy of transportation, is the extent of healthy competition that takes place between suppliers at the fringes of market areas. (Delivered price arrangements and basing-point systems involving conditions of monopoly, oligopoly, or monopolistic competition present issues which are not considered here. It is sufficient to say they do not vitiate, nor detract from, the main thesis presented.)

Transportation and Land Use

Transportation has a profound effect upon land uses and values. Some figures of the 1850's are available showing that a farmer would receive $48 a ton net for wheat if his land was only 10 miles from the major market, but the net price to a farmer 300 miles away would be only $4½—less than 10 percent as much. The difference was the cost of transportation by road. Now, if the direct costs of producing wheat at the "distant" point exceeded $4½ a ton, land there would have had no value for wheat production. However, if the direct costs were $2 at both points, land rent (for an acreage on which a ton of wheat could be produced) would have been $46 per ton at the closer point as compared with $2½ at the distant point. The value of the closer land would have been more than 18 times that of the distant land.

Today, fertile lands in many areas of the world still stand idle because of prohibitive transportation costs. A reduction in the cost of transportation would tend to increase land values in the remote regions but, at the same time, more land would be brought under cultivation and prices would tend to fall.

Many interesting phenomena in land use can be explained by analysis of transportation conditions. In the early days, whiskey production became an important industry west of the Alleghenies; it was too costly to ship wheat except in bottled form. Illinois and Iowa both produced great quantities of corn, but hog-raising became a much more extensive industry in Iowa; while it was profitable to ship corn to market from Illinois, the higher transportation costs from Iowa made it necessary to convert corn to meat prior to shipment.

Examples of the profound and generally beneficial economic effects of improved transportation are almost limitless in number and variety. For instance, urbanization and the later phenomenon of suburbanization would have been impossible without adequate transportation. The main point to be emphasized, however, is that good transportation serves to reduce the costs of production, even though it is itself a cost of production.

Passenger Transportation

A comment or two should be added with respect to passenger transportation. That large portion which is indulged in for business purposes is subject to the same economic considerations already suggested. For example, it permits a greater technical division of labor and thereby promotes efficiency in production. On the other hand, personal transportation for pleasure is consumption—the direct satisfaction of human wants. As in the purchase of any other goods or service, the public interest is to get it as cheaply as possible.

A General Conclusion

What has been said is intended primarily to establish not only that transportation is indispensible in a progressive society, but also that any lowering of the costs of transportation is good for the general economic welfare. Transportation is one of the costs of production. Whenever any of these costs is reduced our economy is strengthened; our standards of living can be raised. It is vitally important, however, that we bear in mind a clear concept of economic costs. By lowering the costs of production we mean either (1) producing the same amount of goods and services with a lesser expenditure of economic resources—labor, capital, and land, or (2) producing a greater amount with the same expenditure. From this it follows that when we appraise the costs of transportation service we must include all of the economic costs. It matters not whether the costs are met publicly or privately. Unfortunately, a prevalent tendency in some quarters is to ignore such costs as highway improvement, navigation development, or airport provision when they are paid for by government—a subject about which more will be said.

ODAY'S TRANSPORTATION SYSTEM

We have in this country a highly developed nd extremely complicated system of domestic ransportation, consisting in the main of railoads, waterways, highways, pipelines, and irlines. A few figures, taken more or less at andom, indicate the significance of transporation in the United States. In 1950 it was stimated that intercity freight traffic mounted to 1,017 billion ton-miles. That gure may mean more if we realize that we noved in intercity transportation 1 ton of reight about 6,800 miles for every man, roman, and child in the country. In the ame year, our people traveled an estimated 00 billion passenger-miles in intercity movenent—the equivalent of 2,600 miles for every erson. The movement of people and goods ithin cities adds substantially to these gures.

We have built up a tremendous private and ublic investment in transportation facilities. Iore important, however, is the amount we re expending annually for transportation. 'he figures are astounding. It is estimated hat expenditures for personal transportation n 1950 totaled almost $23 billion—almost 12 ercent of our total consumption expendiures—and that figure does not include any xpenditures for transportation service that vas incorporated in the prices of things we ought. The combined freight revenues of Class railroads and Class I motor carriers of property lone was in the neighborhood of $12 billion n 1950. For the development and promotion f all kinds of domestic transportation the 'ederal Government alone is spending about billion dollars annually. We need not oncern ourselves too much with figures of his sort. But we should realize that about ne-fifth of our total national effort is being xpended for transportation service—an appreiable slice of the national economy.

Highway Transportation

Of particular interest in this discussion is ighway transportation. The motor vehicle as virtually revolutionized the industry. Barely more than 50 years ago one of our eading economists could say: "The road sysem as a matter of national importance is a hing of the past." Today in excess of 50 nillion motor vehicles—9 million of which are rucks and busses—are operating on our highvay plant. In 1950 we traveled more than 37 billion passenger-miles and carried 126 illion ton-miles of freight over the highways etween cities. We consumed almost 36 illion gallons of motor fuel on our roads and treets. We had about one passenger car for very 3.7 persons and more than one truck for very 18 persons in 1950.

The magnitude of finances involved is peraps more important. Retail sales of autonobiles and automotive products exceeded 26 billion in 1948—more than 20 percent of ll retail sales in the country. It has been stimated that the total annual cost of highvay transportation is somewhere in the neighorhood of $40 billion—about one-seventh of he gross national product. One worker out of seven is employed in some phase of highway transportation.

Total expenditures for highways are almost as astonishing. We are attempting to maintain and improve a road and street plant of more than 3,300,000 miles. In this effort we have spent more than $60 billion for highway construction and maintenance in the last three decades. In 1950 we spent more than $4 billion for highway purposes, of which $2.4 billion was for construction and rights-of-way for State highways and local roads and streets. We collected more than $2.6 billion from highway users in the form of State road-user taxes, and large additional amounts in Federal excises and other general taxes.

Competition in Transportation

Highway transportation has changed a situation of virtual monopoly to one of very considerable competition. The competitive relations among carriers are a matter of basic interest to the economist. It is difficult to get a good grip on this problem, however, for each of the carrier groups has distinctive characteristics. In important areas they are complementary rather than competitive. For example, trucking within cities does not compete with rails or other modern forms of transport. Most trucking from farm to market or railhead is not competitive. Even if certain goods are moved by rail or water or air one time or another, at some time—probably many times—during the flow from producer to consumer the goods are moved by truck. Many such movements are not competitive.

Yet there is no doubt that truck transportation is making serious inroads on rail transportation. In 1950, trucks hauled 12.4 percent of the total ton-miles of intercity freight. The rails hauled 60.4 percent, and water, air, and pipelines accounted for the rest. As between trucks and rails, the

Transportation service brings raw materials to the factory (secondary road in Oregon) . . .

. . . and distributes finished goods to the market. (Maine Ave., Washington, D. C.)

percentages were 16 for trucks and 84 for rails.

The railroads themselves recognize that only a fraction of all trucks are actually competitive. They developed figures for 1948 indicating that only 400,000 of the more than 7 million trucks then in operation—those they called the "highway freighters"—were truly competitive. However, it was also estimated that these vehicles carried about 72 percent of the intercity truck traffic freight ton-miles. This would indicate a total of about 63 billion ton-miles. In the same year, the rails carried 643 billion freight ton-miles—over 10 times as much.

To carry all of the freight traffic in trucks, assuming it were at all possible, would seem to require about 4,400,000 "highway freighters." One may wonder what sort of highway system would be required to accommodate 11 times as many large motor freight carriers as we now have. But all this is mere speculation because there are sound economic reasons why trucks could not handle all the bulk, low-value, long-haul traffic that is today moving by rail and water. It would seem that any talk about decadence of the railroad industry, as if it were on the way out because of trucking, is premature—to say the least.

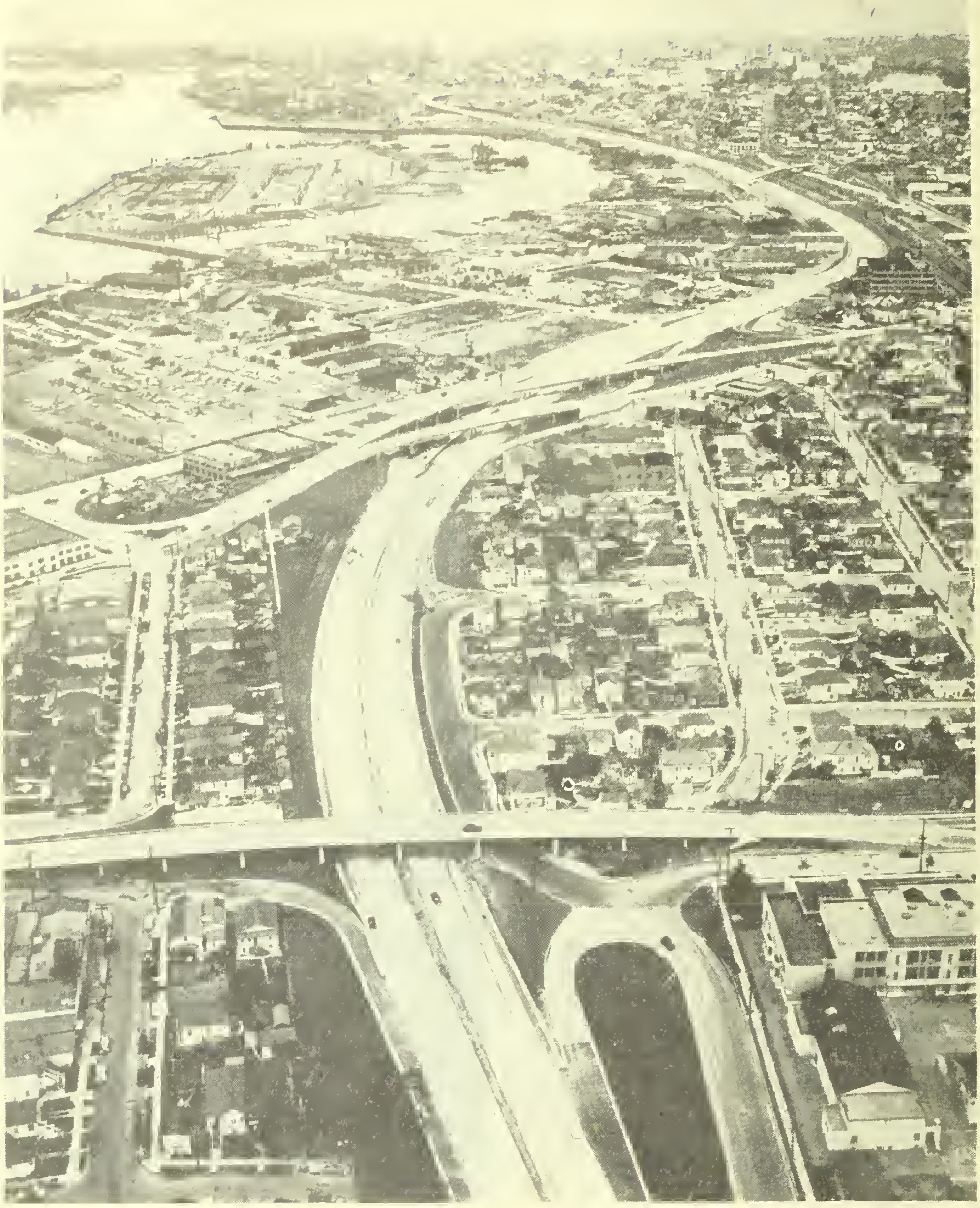
Trends in Transportation Competition

The real significance of highway transportation is not measured by the relative magnitudes, however. The trends are the important thing. For example, in 1940 there were 4,900,000 trucks on our highways. The 1951 estimate is 9,100,000—nearly double. The sizes of vehicles, the loads carried, and the distances run have also increased phenomenally. The Bureau of Public Roads estimated that loaded truck combinations carried 91.4 billion ton-miles of freight in 1950 on main rural roads, as compared with 13.7 billion in 1936—an increase of 565 percent. On the other hand, single-unit trucks carried 14.3 billion ton-miles in 1936 and 29.6 billion ton-miles in 1950, an increase of 107 percent. Average weights of loaded combinations on the main rural roads increased 55 percent between the 1936–37 period and 1950; average weights of loaded single-unit trucks increased 11 percent in the same period.

Here is another important trend. The revenues of steam railways were $4,050 million in 1939 and $9,587 million in 1950, but the revenues of motor carriers of property subject to regulation of the Interstate Commerce Commission increased from $792 million in 1939 to $3,737 million in 1950—an increase of 372 percent for trucks as compared with 137 percent for rails.

A matter of grave concern to the railroads is the fact that rail revenues went down significantly in 1945 and 1946 but in these years motor carrier revenues continued to increase. Again, in 1949 there was a significant 11 percent drop in rail revenues from the preceding year, but the revenues of motor carriers of property increased 8 percent.

One of the important facts revealed by the comparative data is this: Although ICC regulated motor carriers of property in 1948 hauled only 6.6 percent as much freight as rails in terms of freight ton-miles, their revenues were 34.7 percent as much as the rails. In 1950 Class I motor carriers received about 5.2 cents per ton-mile as compared with about 1.3 cents for rails. Thus, it is clear that truck traffic is considerably more remunerative than rail traffic on a freight ton-mile basis.



Savings in operation may justify substantial investments in highway improvement. (Eastshore Freeway, California)

Now, there are several reasons for this. In general, motor carriers carry more valuable, higher-rated commodities; rates for less-than-carload traffic are higher than for carload traffic; rates for long hauls are lower on a ton-mile basis than for short hauls. To the extent, however, that trucks drain off the cream of the traffic, the problem of the rails is intensified, for they are left with the less remunerative traffic which may little more than cover out-of-pocket costs. A continuation of this trend would have a serious impact on the rate structures and financial stability of rail carriers.

Advantages of Highway Transportation

Little time need be spent here in exploring reasons for the phenomenal growth of for-hire highway transportation. Its obvious advantages are flexibility and adaptability and these add up to economy and speed. Of the two, speed—the time elapsed from initiation of the shipment to delivery at the destination—is the more important in many current situations, for time in transit can be quite costly to both consignors and consignees. More rapid transport has fostered the smaller inventory policies of many businesses and has resulted in substantial interest savings. Another of the inherent advantages of highway transportation is the ability of the motor carrier to adapt the size of the vehicle to the shipment, thus making the industry especially capable of handling less-than-carload freight expeditiously. Door-to-door service, less handling of goods, lower packing costs, are among the many other factors that contribute to the success of the trucking industry.

Another factor partly responsible for the initial success of trucking was certain rate-making policies that had been well established while rails had a near monopoly of the traffic. For example, predominant emphasis on value-of-service rather than on cost, in fixing rates, and preferential treatment of long-haul over short-haul traffic, provided a situation in which a new group of carriers, already having certain inherent advantages, could rather easily attract a substantial amount of the more remunerative business. Not to be overlooked, also, is the fact that the motor vehicle provided an opportunity for producers to haul their own goods whenever rates seemed out of line or it seemed convenient to do so. The significant fact is that the development of highway transportation has transformed an industry of monopolistic characteristics to one of intense and effective competition—a condition that the general public and policy makers have been somewhat slow to recognize and appreciate.

An outstanding feature distinguishing highway from rail transportation is the public provision, financing, and management of roadways for motor vehicles. This has made it possible for many thousands of operators to enter the transportation industry. One truck or bus is all that is needed to set up in business. The result has been not only competition between rails and motor carriers, but also intense competition among segments of the motor-carrier industry; and hanging over the entire public-carrier industry is the constant threat of private operation of motor vehicles by industrial and commercial enterprises. The motor-carrier industry, despite the development of a few large carriers, is still characterized by thousands of small operations, involving relatively small investments, and often managed on a family or individual proprietorship basis. Progress has been slow in adapting public policies to the realities of this new situation. The regulatory agencies and the courts are just beginning to discard the monopoly concept and come to grips with the problems of "regulated competition."

HISTORICAL ASPECTS OF THE HIGHWAY FUNCTION

The rapid development and wide-spread use of the motor vehicle, which has so enormously increased the capacity and efficiency of our transportation facilities and changed the basic character of our transportation system, has also brought with it a whole new set of problems, particularly involving questions of public policy.

The enormous growth of motor-vehicle use sometimes tends to obscure the historical significance of roads. But in any fair appraisal of highway transportation today, it is essential to bear in mind that highways were not provided publicly in direct response to the demands of the motor-carrier industry. Nor, for that matter, is the motor vehicle responsible for the initial development of the road plant.

An interesting story, fraught with both economic and social interest, can be traced in the evolution of roads from the footpaths of primitive societies, which followed animal trails, to the modern high-speed, controlled-access expressways of today. We can only mention in passing the network of roads developed by the Roman Empire which contributed much to its military and other successes; and, on this continent, the roads built by the Incas of Peru which not only surprised the European explorers who found them, but are regarded even today as substantial engineering feats in view of the physical and technological handicaps under which they were constructed.

Early English Roads

Among the interesting features in English road history is the extensive network constructed by the Roman conquerors between A. D. 43 and 407, about which one writer commented that no engineering work comparable was seen until the railroad era, and another said, the history of English roads is almost the history of how and why the Roman roads became the modern trunk roads.

Another English development, perhaps having current significance, was the spread of a system of private toll roads, called turnpikes, in the 18th and early 19th centuries. These roads were privately constructed under governmental authority and protection. As a result of more than 4,000 legislative acts, all of the main roads of the country became toll roads and there was great improvement of the general highway system. Here then, is perhaps the first example of commercial operation of the road plant, based upon the concept that highways are not merely of local interest but are so important to the users that special charges can and perhaps should be imposed for their use. Public dissatisfaction with the toll barriers led to a process of "disturnpiking" inaugurated in 1837 but not finally completed until 1895.

Early American Roads

As might be expected, road history in early America rather closely paralleled the English history of the times. The first half of the 19th century was mainly a turnpike era. In 1792 Pennsylvania chartered the Lancaster Turnpike Company which constructed the first macadam road in the country as a toll facility. The toll movement spread rapidly. New York had chartered 500 toll bridge and turnpike companies by 1836; Pennsylvania had 220 companies; New England was covered by a network of toll roads. (And some people today regard the toll road as a modern innovation!)

A great many speculative excesses and abuses as well as mismanagement of the toll roads led to financial failures, resulting in losses both to individuals and to States that had heavily invested in them. Toll roads began to disappear from the scene about the middle of the 19th century, not only because of mismanagement but also because first canal and then railroad building had captured the public interest. So it was that highway improvement and maintenance became an orphan, to be sadly neglected during the latter part of the 19th century. What little was done, was done by local government, often under a procedure by which the citizens were expected to work out their local taxes by working on the roads, a practice that was characterized by inefficiency but seemed to be rather enjoyed as a social event. It was under these circumstances that Arthur Twining Hadley (then president of Yale University) could say in 1900 that roads no longer had national importance.

As a matter of fact, however, a good-roads movement was beginning to make itself felt in the late 1800's, before the automobile assumed any real importance. Railroad interests and bicycle manufacturers as well as farmers spearheaded the movement. But the real impetus was to come after the motor vehicle attained a significant place in the transportation scene.

Competition, Conflict, and Change

Perhaps one of the lessons to be learned from the history of transportation is found in the great conflicts of interest that arose whenever new methods appeared on the scene. The turnpikes gave way to canal and railroad building. The significant fact is that efforts were made to perpetuate the turnpikes; again much effort was devoted to the protection of canals when railroads promised to become a threat. Even the conflict between "wheelmen" and "livery men" at about the turn of the century was intense enough to evoke comment from one of California's first highway commissioners to the effect that every good bicycle town was also a good livery town so the livery people should not stand in the way of road improvement.

In our competitive society the old is expected to give way or to be modified by the new; it is to be expected, also, that vested interests in the old will resist change and bend every effort to protect themselves from new competition. Yet our economy is founded on change, however ruthless it may seem, for it is based on the principle that the exercise of intelligent self-interest nearly always advances the public interest. One who understands this will not be too surprised at the current campaigns of opposing forces in the transportation industry. He will understand why such fights often generate more heat than light and that the claims and counterclaims on all sides are frequently excessive. But even a dispassionate appraisal of the current situation will reveal that there are significant problems in transportation which do affect the public interest, even if they do not add up to the "crisis" one so frequently hears about.

The carrier groups are more complementary (Chicago airport) . . .

. . . than they are competitive. (Washington, D. C.)

THE HIGHWAY FUNCTION TODAY

The States today are managing what in effect amounts to one part of a full-scale transportation service. There has developed a partnership in transportation, not often fully appreciated, between government, which supplies the roadways, and private individuals and business firms, which supply the operating equipment. It is no easy task to coordinate these elements into an efficient transportation service. One of the great difficulties arises from the present-day conflicts of interest among various groups of users of the publicly provided road plant—for example, between truckers and automobile owners and among competing commercial highway users, such as common, contract, and private carriers. Additionally, there is the bitter controversy between motor carriers and other carriers, such as the rails.

In analyzing the transportation problem of today, one fact is worth remembering. The basic outlines of the American highway system were laid out long before any competitive threat of commercial motor-vehicle transportation to rails or other transportation agencies was realized. And the highway problem of today cannot be laid on the doorstep of the half-million heavy trucks that offer direct competition to other transportation.

The accommodation of more than 50 million other vehicles—passenger cars and complementary commercial vehicles—poses the basic highway problem. The freedom of movement, convenience, speed, and other features of private passenger-car transportation which have appealed to the American people and have led to the expenditures of a substantial portion of their incomes for highway transportation are largely responsible for modern highway development. One evidence is that, of the 400 billion passenger-miles traveled in intercity movement in 1950 by all forms of transportation, almost 85 percent took place in private passenger cars. Recognition of this situation, however, should not obscure the economically important fact that highways are multiple-purpose facilities which are used by many kinds of vehicles.

Economics of Joint Highway Use

The fact that highways are publicly provided and jointly used is perhaps one of the greatest economic advantages of highway transportation. The road plant is supplied in response to numerous and varied demands. We have, therefore, an outstanding example of the economies of joint use, on the one hand, and the problems of joint-cost apportionment, on the other.

To illustrate: A basic highway plant may be provided for passenger cars under any circumstances; if trucks are permitted to operate on the same highways and pay all special costs for which their operation is directly responsible, any additional payments they may make go toward defraying the costs of the basic highway, and hence benefit all passenger-car operators by reducing the costs of the basic highway to be apportioned among them.

Conversely, the truck operator who pays all of the direct highway costs assignable to him is benefited by the tax contributions of passenger-car operators that go to defray the overhead costs of the basic highway plant.

The point is that when different groups of users can use the same highway plant both the total costs and the costs for each group will be less than if each were required to supply its own facilities; provided, of course, that each group meets all costs for which it is directly responsible and contributes something to defray the joint or overhead costs of the plant.

Perhaps a purely hypothetical example will help at this point. Suppose that a given stretch of four-lane highway adequate for passenger cars only could be constructed for $200,000. Suppose the same stretch could be built to accommodate both passenger cars and trucks for $240,000, and that a two-lane highway costing $140,000 would suffice for trucks alone. Now if two highways were built—one for passenger cars and one for trucks—the total outlay would be $340,000 as compared with only $240,000 for the single highway that would carry both kinds of traffic. The loss to the users—and to the economy—would be $100,000. Now if, for this single highway, passenger-car operators paid anything less than $200,000, they would save money. By the same token, if the truck operators paid anything less than $140,000 they, too, would save. Thus, if passenger cars were taxed $150,000 and trucks were taxed $90,000 each group of users would save $50,000 over what it would cost to supply themselves separate facilities. Here, then, we see clearly the economies of joint use.

Problem of Cost Assignment

But, at the same time, the example illustrates the problem of highway cost assignment. In the first place, it might be difficult to prove that it would cost $40,000 more to provide the highway for both kinds of traffic than for the passenger-car highway alone. And, if that hurdle is passed, the problem remains of apportioning the joint cost—that cost for which neither group can be held solely responsible—between them. Shall passenger cars be charged $200,000—the amount required for a basic highway? If so, the trucks would be charged only $40,000. Or should truckers be charged $140,000—the amount they would pay for their own highway? If so, passenger cars would be charged only $100,000.

The point is that anything the truckers pay over $40,000 helps the passenger-car operators, and anything the passenger-car operators pay over $100,000 helps the trucks. The complexity of apportioning joint or overhead highway costs among users becomes obvious. Even assuming that all direct costs have been ascertained and assigned to the responsible groups, all that can be said for sure is that truckers should pay at least $40,000 and not more than $140,000, while passenger-car operators should pay at least $100,000 and not more than $200,000.

Thus, to apportion the truly joint costs of the highway plant in order to fix reasonable user charges, we must turn to some theoretical basis such as relative use, differential benefit, or value-of-service. Naturally, there is much room for disagreement. The situation is tremendously more complicated in actual practice than in our example. First, we lack the detailed information necessary to ascertain to anyone's satisfaction the direct costs of the entire plant for which any particular group of users should be held responsible. Second, we are dealing with many kinds and sizes of vehicles, carrying all types of passengers and commodities, using different segments of the highway plant in varying degrees, operating different mileages during the year, and, by administrative necessity, subject to different kinds of imperfect user-tax bases.

Various Concepts of the Problem

The manifold problems involved in rationalizing and coordinating transportation service today are overwhelming. Anyone familiar with the subject would readily subscribe to the words of the late Director of the Highway Research Board, Roy Crum, when he said: "To visualize the whole picture at once has already given me mental indigestion."

One of the basic difficulties is to get and retain perspective. Not only among the directly conflicting forces are differences of opinion found. Many significant differences in approach to highway and other transportation problems are found among observers who presumably are impartial or at the least should not be passionately interested in the outcome.

For example, the highway engineer may be primarily interested in getting roads built to accommodate traffic; the competitive aspects of highway transportation do not trouble him; even the questions of wise allocation of limited economic resources or equitable taxation for highways are likely to be given short shrift.

The transportation economist, on the other hand, is likely to emphasize to the exclusion of all other considerations the competitive aspects of highway transportation and the effects on other carriers; private transportation, the prime generator of highway demand, may be virtually ignored.

The public-finance expert may fail to recognize the competitive aspects of highway provision and treat it as any other function of government, to be financed under the general tax system; he may see no merit in legal provisions against diversion to nonhighway purposes of taxes collected on a user basis; he is likely to advocate annual budgeting of highway funds in the same manner that all other funds are budgeted. He fails to appreciate what the transportation man sees or to recognize the engineer's demand for a reasonably steady and assured flow of income in order to permit long-range planning and orderly development of the highway system.

Such circumstances remind one of the parable of the blind men whose descriptions of the elephant differed so much because each had come into contact with different parts of its anatomy. If the experts do not see eye to eye, but get "mental indigestion" when viewing the whole transportation problem, is it any wonder that laymen have difficulty? Is it any wonder that our policy makers find little positive guidance in reaching the practical decisions they are inexorably forced to make?

Divergent Principles

Perhaps one source of difficulty in the management of our highway affairs lies in what Professor Shorey Peterson of the University of Michigan has called our "vacillating allegiance" to the divergent principles of the public economy and the private economy when confronted with various issues.

On the one hand, we are prone to treat highways like any other function of government. We do not distinguish taxes for highways from other taxes. We rely upon political judgment as to how much to spend for highways just as we do in determining how much to spend for schools or welfare. We also rely upon judgment as to how to distribute the cost, with little regard to the possibility of differential benefits to different groups or individuals. We justify expenditures in broad generalities, stressing the general welfare.

One unfortunate aspect of this approach to the highway problem is that we are left without any effective guides either as to the proper amounts and rates of investment in highways or as to appropriate methods and levels of taxation. Our sole criterion is how much we can collect from the taxpayers. When that has been determined, we must decide between many worthy but conflicting claims to the tax dollars on the basis of some vague, almost intuitive feeling of the effect upon the general welfare of alternative courses of action.

Operations in the private economy are, of course, governed by quite different considerations. Basically, the laws of supply and demand prevail. Goods are supplied in response to demand; the individual indicates the benefit he expects to derive from the various goods and services by the prices he is willing to pay. On the other side of the equation, goods and services are supplied if the prices offered will meet the costs of production—the costs of the factors, labor, capital, and enterprise, that go into them. Thus, in effect, the demand-supply equation becomes a benefit-cost equation. It has been suggested that principles applicable in the private economy can be adapted with appropriate modifications to serve as a guide in dealing with basic highway issues.

The motor carrier is adaptable to the commodity in shipment. (U S 1 north of Washington, D. C.)

A Modified Commercial Concept

As we have seen, with the exception of the comparatively brief experience with toll roads, the provision of highways was treated as a general function of government before the advent of the automobile. The development of the motor vehicle has modified our thinking. The extended radius of travel, the types of roads required, the magnitude of finances involved, the competitive aspects of highway provision in a private-enterprise economy, the need for standards governing the rate of highway investment, all conspire to give the highway function a much deeper economic significance than it formerly had. That the approach to highway problems in terms of the private economy has made headway is evidenced by the elaborate user tax systems founded on a benefit concept that have been developed and revised from time to time in all States. Further refinement and application of this commercial concept to highways may aid in solution of the perplexing problems of cost allocation and investment.

ALLOCATION OF HIGHWAY COSTS

It has already been suggested that the allocation of highway costs among the various beneficiaries is one of the basic issues in highway finance. One could engage in an involved and extended discussion here, which would lead deeply into the complicated and controversial subsidy issue. What needs to be emphasized is principle. There are good reasons why users should not be held responsible for all costs of all segments of the highway plant—a certain share should be assigned to property owners and general taxpayers. Once an assignment of responsibility to the highway users has been made, however, they should be expected to defray through user taxes all of the economic costs associated with highway provision.

The second difficult cost assignment problem is the determination of shares of responsibility to be borne by vehicles of different sizes and operating different mileages on the highway. Perhaps no other issue is so bitterly debated. On the one hand, one finds the claim that heavy vehicles are pounding the pavements to pieces; on the other, that properly built highways are unaffected by heavy vehicle use. At one extreme, it is contended that fuel consumption is a suitable measure of highway use; at the other, that weight and distance as reflected in the ton-mile is the best measure of relative highway use and, hence, of highway benefit.

Conclusive Answers Lacking

The very fact that there is so much dispute suggests that conclusive answers have not yet been found. This is the case. Various theories have been proposed for spreading highway costs. Just to name them—differential or incremental costs, relative use, value of service, differential benefits, differential space occupancy—is enough to suggest their complexity and diversity. It is perfectly clear that one cannot hope to find a formula upon which everyone can agree. For one thing, too little is known about highway costs. This may seem disconcerting in view of the fact that many billions have been invested in highways, but actually the problem is as much economic as it is engineering in nature.

Before we are too critical of this situation, we should recall that in many other fields where difficult overhead cost problems are met no precise solutions have been found. As one example, transportation rates for different commodities and distances are based largely on value of service—that is, what the traffic will bear—rather than on cost. All direct costs of providing transportation service have not been isolated and measured; even if they were, the large remaining portion would be truly joint or overhead costs that would have to be distributed upon some theoretical basis. Price fixing for highways is analogous to, and subject to the vagaries of, rate making for utilities.

There is, of course, a basic difference between private investment for profit and public investment, even when an attempt is made in the latter case to adopt certain principles applicable in the private economy. It is not to be inferred that the highway plant ought to be operated for profit. As in other monopoly situations, prices for highways might be fixed to maximize profits. However, it is well accepted that unregulated monopoly profits ought not be tolerated even in a private enterprise economy. Fixing charges for highway use is somewhat analogous to fixing rates for utilities where the purpose is not to maximize profits but to cover all costs plus a fair return on the investment. In various highway situations, of course, there may be a consumers' surplus in the sense that users would be willing to pay more than would be required to meet costs.

It seems clear that long-term research and more concentration of attention on the highway problem by people of diverse but relevant competences will lead to better solutions of the cost assignment problem than we now have. That the problem is receiving the interest it warrants is indicated by certain recent developments, among them being the creation by the Highway Research Board of a number of subcommittees "to bring about a frontal attack on this problem," each of which will deal with different phases of the question of equitable allocation of highway-tax responsibility. The Maryland test road and other studies to be operated are expected to provide useful data in this connection.

The essential requirement is that we focus attention on the costs of providing highways and strive toward the assignment of costs among beneficiaries on a rational basis. The goal toward which we press is to set the supply prices of highway service as they might be set in the private economy. When this is done in highway management and other fields where public aid is a factor, a realistic and economical basis for competition among the various transportation agencies will be established; and the rates of investment and distribution of traffic among the carriers will be determined by their relative economy and fitness.

THE MODERNIZATION PROBLEM

Perhaps one of the toughest public issues of the day concerns the appropriate rate of investment in the highway plant. We are informed by highway engineers, and indeed everyday experience tells us, that our highway systems are critically deficient in important respects. A Nation-wide campaign for better roads is under way, sponsored by important segments of the highway transportation industry. All evidence points to the fact that a considerably faster rate of highway expenditure will be required to bring the highway plant up to standards that will be regarded as satisfactory by the great majority of private and commercial users.

Highway Needs

It was estimated by the California Division of Highways last year that more than $3 billion would be needed to eliminate the State highway deficiencies that existed under 1950 traffic and cost conditions. Revenue estimates indicate that to do this job about $1.5 billion—only half enough—will be available by 1962, 10 years hence. And during this period traffic will continue to grow and probably will not generate enough revenue to meet the additional needs as they arise.

Owen and Dearing [1] have summarized the Nation-wide highway situation in these words:

> We have now entered a new era of highway development. This stage in the physical development of the highway system is characterized by technical standards and capital requirements that make previous concepts totally inadequate. Highway administrators are confronted with a situation analogous to that of an entire industry being overtaken by functional obsolescence. Survival depends on modernization; but in order to modernize, the old tools must be replaced and the entire plant redesigned.

In some quarters fear is now developing that a continuation of present highway inadequacies and the prospect of even worse conditions will have a serious impact upon motor-vehicle usage. The consequences would adversely affect the general welfare, not only because of the deleterious effect upon transportation efficiency, but also because of the effect upon manufacturers and purveyors of automotive products. As we have seen, an appreciable part of our economic prosperity in reasonably normal times is dependent upon highway transportation of all kinds. It might be observed at this point that the rails and waterways carry an appreciable amount of freight that is dependent upon a thriving highway transportation industry—not only finished products but also the materials that go into motor-vehicle production and highway building and use.

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[1] *Toll Roads and the Problem of Highway Modernization,* by Wilfred Owen and Charles L. Dearing. The Brookings Institution, 1951, p. 23.

There is conflict of interest among various road-user groups, but joint use is one of the advantages of highway transportation. (U S 1 at Cottage City, Md.)

Public Apathy

In spite of the evidence pointing to great highway inadequacy and a vague general realization of the fundamental importance of highway transportation to the national economy, there is considerable reluctance among the public to meet the issues squarely in the one way that really counts—additional financing. Perhaps an educational campaign with dramatic appeal and readily understandable arguments might dispel this public apathy. The fact is that a bare inventory of highway deficiencies, regardless of its size, does not in itself make a compelling case for additional financing or furnish a satisfactory guide either as to the aggregate amount that should be expended or as to the rate of annual expenditure that might be tolerable.

Time and again I have heard the heads of business state that their department heads or engineers can supply them with statements of needs and worth-while expenditures so large in the aggregate that they all cannot be financed, however much they might improve the operations. Almost every family can make up inventories of justifiable needs that add up to expenditures far in excess of their incomes.

The stark reality is that both the businessman and the family must adjust their expenditure programs to what they can afford out of current income or what they decide may be so essential as to justify pledging future income by borrowing. The businessman's decision is influenced not only by the demand situation as he estimates it, but also by the alternative uses to which he might put the limited capital available to him. The family must inevitably balance benefits from one possible outlay against benefits that would be derived from alternative expenditures.

It is not so with government in its usual operations. The government's power to tax gives it the power to adjust income to the expenditure program that is regarded as necessary or desirable. The crux of the problem is to determine what is necessary or desirable. Ordinarily there are no positive guides and so we rely on the collective judgment of the people as expressed through their legislators. But in the case of highways, that collective judgment may be influenced and perhaps improved if we can apply at least some of the standards and principles that would apply in the private economy. What is needed is an accurate evaluation of the benefits of improved highway service, and a realistic comparison of those values with the full economic costs of providing the service. This guiding principle is much easier to state than to apply, however.

Popular Appeal of Toll Roads

Perhaps one of the reasons that toll roads are attracting interest and active support in many circles is that a value-cost comparison is applied in a way that can be generally understood. The individual user can establish for himself a value-cost relation; before every trip he can weigh the benefits of using the facility against the toll charges. Ordinarily, the financial solvency of a toll facility is subjected to rigorous engineering and economic analyses before it is undertaken. In effect, the potential income depends upon the estimated demand for the service which, in turn, reflects the users' estimates of the benefits they expect to receive. This potential income must be sufficient to meet the costs of the project, including interest and an extra margin for the risk involved, before the project will be approved.

Unfortunately, no effectual way to apply this sort of value-cost analysis to the entire highway plant has been developed. Important applications of the value-cost ratios are made in determining relative merits of alternative highway investments in the effort to maximize benefits to the highway users. However, the problem is conceptually different from that under discussion here, for the money is already available for one highway improvement or another; the die is cast, the issue is not between highway expenditures and alternative unrelated public or private uses of the funds.

Without, at this time, judging the merits of toll financing under certain conditions, we may say that the basic highway problem does not permit solution by the erection of toll gates everywhere. This means that taxation must be used. As a general rule, however, highway taxes must be uniform throughout a political jurisdiction and must apply to everyone using any segment of the highway plant. What the people would pay based upon individual evaluations of the benefits if they had the choice of using or not using particular highway facilities will not be known as it is in the case of toll financing. But as a guide to policy it might be possible to approximate a hypothetical demand curve based upon an estimate of benefits that highway users might expect to derive from a particular level of highway improvement.

Measurement of Highway Benefits

One obvious clue to the measurement of benefits to users is the effect of highway improvements on the total costs of highway transportation. As a matter of fact, we are spending considerably less than 10 cents of our highway transportation dollar for the road plant. A larger percentage spent on highways may mean a lesser total cost. This is the real import of the thought that we pay more for poor highways than for good ones.

For example, savings in distance resulting from a highway improvement may result in savings in fuel and tire costs alone that will be more than sufficient to pay for the improvement. Less congestion, better road surfaces, reductions in grades all tend to reduce vehicle operating costs. In each case the benefits to the users can be reduced to monetary terms and offset against the highway cost.

Moreover, savings in time that may result from any one of a variety of highway improvements may have tremendous economic value even if we conservatively consider only the time of commercial and business highway users. The saving of 5 minutes may seem a small matter for one operator, but over the

How shall costs be assigned among vehicles of different sizes, operating different mileages? (U S 1 near Newark, N. J.)

course of the year, during which thousands of vehicles are involved, the savings add up to enormous sums which, when compared with the highway costs, may justify very substantial investments in highway improvement.

The reduction in accidents that results from highway modernization also means lower costs for both commercial and private users, not only in the savings through elimination of accidents but also possibly through reduction of insurance rates.

In addition to tangible savings in operating, time, and accident costs there are intangibles to which, unfortunately, money costs cannot be attached—as, for example, the increased pleasure and comfort of the motorists on better highways. Undoubtedly these are benefits for which a great many motorists would also be willing to pay. It would seem proper, therefore, to reduce benefits to money equivalents wherever possible. But when the final decision is to be made, the intangibles must be evaluated by subjective judgment and may tip the scales in favor of higher level of improvement than might be indicated by monetary values alone. Here, then, after the technicians have completed their analyses, the collective judgment must come into play in reaching the final decision.

The Users' Willingness To Pay

Concrete evidence of the highway users' willingness to pay for improved highway service is found in the cases of certain toll roads in the East, where existing parallel non-toll roads are worn out, obsolete, or inadequate. Motorists usually pay about 1 cent per vehicle-mile for operation on these toll facilities—the equivalent of a gas tax of 15 or 20 cents a gallon in addition to their ordinary user taxes. Obviously the benefits of the higher type facilities, both tangible and intangible, as evaluated by the users themselves are substantial.

The case is even more striking when commercial and business uses alone are considered, for here the intangibles play a lesser role and the cold calculations of cost against value-of-service are dominant. Yet it appears to be to the advantage of the trucking industry to pay substantial tolls. For example, trucks are charged from 2 to 6 cents per mile, depending on size, for operation on the Pennsylvania Turnpike, yet about two-thirds of the revenue is derived from truck traffic. The reasons are apparent. Studies indicate, for example, that fuel consumption of a 50,000-pound gross-weight combination is 50 percent higher on an alternate route than on the Turnpike, and more than twice the time is required to travel the same distance on the alternative highway.

Problems of Estimating Costs and Benefits

We have alluded from time to time in this discussion to the need for including full costs in appraising the relative economy of highway transportation. Without further elaboration, it seems self-evident that all costs must be included in any value-cost comparison that is to be useful in solution of the highway investment problem.

It should be mentioned that this consideration is generally based upon the assumption of an economy of full employment. A departure from value-cost calculations, in the rigid sense, may be justified in a situation of under-employment. At such a time accelerated highway improvement may be considered an appropriate device to relieve unemployment and stimulate the economy. Even under such conditions, however, it is appropriate to maximize the benefits of highway improvement; on the cost side of the equation it will be appropriate to include less than the full monetary outlays (or perhaps calculate costs at zero) if, in fact, inflationary methods of financing are used which result in the employment of labor and other economic resources that would otherwise be idle. However, for purposes of comparing alternative public expenditures under such conditions it may be useful to develop benefit-cost ratios in which full monetary costs are included.

It should be emphasized that there are tremendous difficulties in the way of making a precise value-cost appraisal for the entire highway plant. Not enough is known about vehicle operating costs for different vehicles under different highway conditions. It is difficult to estimate the value of time savings for business and commercial users and debatable whether time savings for private users should be evaluated in money terms or regarded as an intangible. Placing an acceptable money value on highway accidents is no easy task, even if a satisfactory method of estimating potential reductions is agreed upon. Nevertheless, it seems clear that efforts should continue in this direction. It is to be hoped that continuing research will reduce the areas in which disputable assumptions or purely subjective evaluations are required.

Highways and the General Welfare

It may have been observed that these remarks have been confined to the benefits that accrue to the highway user. No mention has been made of the social and political values which are frequently ascribed to highways and emphasized in justifying an improvement program. The existence and importance of general benefits cannot be denied. But they do not lend themselves readily to specific economic evaluation. On the contrary, too much emphasis on general benefits may prove misleading. The basic problem under consideration is the diversion of limited economic resources from other possible uses to highways.

In many discussions of general benefits the fact is overlooked that the use of these resources for other purposes, whether private or public, may also render great general benefit to society. Almost every investment results in a general benefit. If it is said that highways aid the national defense, can it not be shown that a steel plant is also essential to the national defense? If it be said that highways weave together the social fabric, or promote political cohesion, or bring new lands under beneficial use, can it not also be shown that railroads or canals will render similar benefits? If general benefit were the basic criterion of Government expenditure, nearly all economic activity should be subsidized. To attempt to weigh the general benefits of highway provision against the general benefits that would be gained from any one of all the possible alternative uses to which the resources might be applied is a task almost incomprehensible in its sheer magnitude.

Yet it must be admitted that there are situations in which highway or other public expenditures may properly be found by collective judgment to be justifiable, even though they do not measure up under rigorous benefit-cost analysis. The benefit-cost approach is no panacea. To illustrate: The public purse has often been opened for the promotion and development of industry; perhaps in no field has its use been more generous than in transportation, starting with early road development and extending to canal building, land grants to railroads, development of inland waterways, and, continuing to the present, airmail subsidies and airport and airway development. Many roads are developed (and more are requested) which cannot be justified on a current traffic basis but which, it is hoped, will open new areas for profitable agricultural, mining, lumbering, or other development.

Quite obviously, the use of tax money for promotions of this sort must rest largely on collective judgment based upon such facts as can be marshalled regarding the future usefulness of an industry or a transportation agency or a newly developed area to the general welfare. Under such circumstances, it is obvious that any attempt to adopt standards applicable in the private economy must be abandoned. The problem is purely governmental in character; general benefit rather than benefit to individuals or identifiable groups of individuals furnishes the standard. It is inappropriate to raise revenue for such ventures through user charges; instead, general tax funds should be drawn upon.

Frequently it will be appropriate to use special assessments or property taxes to defray the costs of such highways, as in the case of streets in new subdivisions or roads in areas opened by reclamation projects. In other cases as, for example, access roads to military reservations, appropriations of general funds raised under prevailing tax institutions would seem appropriate.

Practical Expenditure Problems vs. Theoretical Cost Analyses

It would be nice if the value-cost approach provided a simple answer to the highway investment problems, but it does not. Not only must the question of highway justification on the basis of general benefits be dealt with under certain circumstances, as pointed out above, but other practical problems are encountered in the attempt to fix aggregate highway expenditures for the benefit of users on the basis of value-cost calculations. For example, it is quite proper to use true annual costs which include maintenance, depreciation, interest, and possibly taxes rather than expenditures in the comparison of value and costs. If highway improvements are financed through borrowing, or if, by sheer coincidence, annual depreciation on the highway plant provides adequately for new investment, a showing that benefits are greater than costs is useful.

But a pay-as-you-go system of highway finance, however desirable it may be on the basis of other considerations, may make benefit-cost comparisons virtually meaningless as a practical matter, particularly if the entire highway plant is in need of rehabilitation as seems to be the case today. The highway users are called upon to advance money to meet expenditures for improvements long before the expected benefits accrue. It is true that the inclusion of interest in the cost calculation theoretically solves the difficulty. But as a practical matter, the taxpayers are immediately burdened while the compensating benefits they will eventually enjoy are spread out over a considerable time in the future. Under such circumstances, a strong case for credit financing to spread the highway costs over time can be made.

More elusive is the fact that although interest clearly should be included in cost calculations for certain purposes, it is by no means certain that it should be included in tax charges levied for the use of highways that have been financed on a pay-as-you-go basis. This observation warrants much theoretical analysis that cannot be included here.

Previously mentioned was the fact that economists and engineers view the problems of highway finance from different perspectives. Economists would relate user charges to depreciation, maintenance, interest, and possibly taxes, so that the highway users would pay full economic costs for that portion of the existing highway plant for which they are held responsible and which they are currently using. In this manner, subsidy is eliminated. But engineers are looking to the future. They are interested in an adequate expenditure program to expand the highway plant to take care of existing and anticipated traffic conveniently and economically. The fact is that if only annual costs of the existing plant are met by users the only amounts available for expansion of the plant would be the imputed interest that might be included in ascertaining annual costs. If additional funds are secured by increases in user taxes, users would pay more than annual costs and there would be an overpayment—a negative subsidy.

The difficulty might be solved by borrowing or by the use of general tax funds for highway expansion. In the latter case, the popular notion that the use of general taxes for highways is clearly an indication of subsidy to users should be dispelled. However, the financing of a highway plant through general funds with subsequent repayment by user charges which include interest and general taxes, while conceptually sound in economic theory, would require a radical departure from present thinking about highway problems. It would raise another difficult policy question. Investment of general funds in the highway plant provided for benefit of users would be based, not upon the free choice of individuals, but upon collective judgment as expressed through legislatures and effected through compulsory taxation. What standard would guide policy-makers in determining how much current income or savings should be taken from individuals through general taxation and invested collectively in highways?

Favorable Benefit-Cost Ratio Not Conclusive

The practical problem of dealing in highway expenditures when actually financing highway improvements but dealing in economic costs in making comparisons with highway benefits raises other issues. Perhaps these can be illustrated by a homely example. Let it be assumed that accurate cost studies show that a home freezer will pay for itself over a period of time by direct savings in food costs, and that it will yield other benefits such as savings in shopping time. Establishment of these facts will not be likely to cause every family in the nation to rush out and acquire a freezer. We have suggested one reason: An immediate expenditure is required; the savings will accrue over a period of years. (Even if credit is used for the purchase, the time extended ordinarily will not be long enough to reduce payments to an annual cost basis.)

There is another reason. Each family has alternative uses for its funds. Suppose a family is faced with a choice between a freezer

Our street and highway systems are critically deficient in important respects. (Pratt St., Baltimore, Md.)

and a television set. The annual money costs may be the same. But the cost to this family of acquiring a freezer will be the value (anticipated benefits) of the television set which would have to be given up. In its judgment the television set may yield greater benefits than the freezer, even though it has been proved that the freezer will pay for itself over time.

The import of this observation is that costs as computed in highway benefit-cost calculations for investment analysis may be understated. It might be assumed that highway improvements will be justified when the benefits gained, evaluated in monetary terms, exceed the costs. But a home freezer also may yield benefits that exceed its cost. Just as the family might choose a television set instead of a home freezer, given a choice it might choose either of these instead of highway improvements, even though the latter was shown to yield benefits in excess of costs.

Similar considerations might apply in business situations. Highway improvements may result in savings which over time more than offset their cost for a particular trucking firm. But suppose the firm could gain even greater savings with a similar outlay for modernizing its fleet of vehicles. This would be the more prudent investment, if the firm had any choice.

Again, consider the total economy. Suppose it is proved that a given investment in highways yields benefits that far exceed costs. But society as a whole has alternative uses for its economic resources. And value-cost calculations for a reclamation project may also show benefits exceeding costs. How will the choice be made between the highway improvement and the reclamation project? Perhaps if there were only two alternatives an answer could be devised. But a formula to choose between highway improvements and all other possible uses, both private and governmental, both productive and consumptive, to which the limited resources might be put would be virtually incomprehensible.

Judgment Is Involved

It is clear, then, that benefit-cost analysis is subject to severe limitations, at least in the present state of knowledge. Individuals, businesses, or society as a whole may well find that highway improvements, despite favorable benefit-cost ratios, are less desirable than alternative uses of the resources. In the final analysis, the decision on aggregate highway investment must rest upon collective judgment expressed through the legislature. This is not to say that benefit-cost analysis serves no purpose. Quite obviously, if benefits are less than costs, highway expenditures would not be warranted. But where the benefit-cost ratio is favorable, subjective judgment must come into play. The higher the ratio, the more convincing is the case for highway expenditure. But at some point on the scale, perhaps a cut-off should be established. That point could only be determined by intersubjective agreement.

We have entered a new era of highway development. (Approaches to Harbor Drive, Portland, Oreg.)

Our discussion has been confined to benefit-cost calculations as a possible criterion for highway improvement programs. An approach to the problem, now widely discussed, is based on an engineering evaluation of the sufficiency of various segments of the highway system, each segment being given a rating in relation to an adequate engineering standard. But unless all roads are to be brought up to the adequate standard, a point must be selected below adequacy which can only be determined by subjective judgment. A possible guide in making this judgment is a comparison of highway benefits with costs at various points on the sufficiency rating curve.

The fact that no formula has yet been devised to grind out a precise answer to the problem of highway investment is not surprising. Wherever economic choices are required, individual or collective judgment is involved. In the private business world, final decisions between alternative courses of action rest upon judgment of the head of the business or the board of directors. The economic decisions of the individual, the family, the business, and the government need to be buttressed by facts. One of the basic aims of economic research should be to supply the facts and make the analyses that will lead to more rational decisions.

With respect to highway finance, there may be no substitute for informed judgment, but as an initial step forward we need merely obtain public recognition of two basic concepts: (1) that highway transportation efficiency and economy depend upon both the public roadway and the private vehicle, and (2) that highway expenditures should be evaluated by weighing benefits against cost, and not as government largess. The results cannot fail to be salutary.

CONCLUDING OBSERVATIONS

Surely by now it must be clear why a contemplation of highway transportation economics is enough to give one "mental indigestion." We may take comfort from the fact that, despite all the difficulties and confusion, we have developed a remarkably effective and efficient transportation system in this country. There is every reason to believe that further progress will be made. Competition will continue; there will be shifts of traffic; the old methods will give way to the new; and over-all efficiency will increase. It is perhaps inevitable in the competitive system, particularly when complicated by varying degrees of governmental participation, that inefficiencies and imbalances will be found when comparisons are made with theoretically ideal conditions.

There is no easy way out of the difficulty, for it is apparent that the fundamental issues would remain, and perhaps be greatly exaggerated, even if our transportation agencies were unified under complete government control. Fortunately, we can afford to tolerate some economic losses to gain the greater advantages that accrue under the stimulus of competition which leads constantly to new developments and improvements. It is to be hoped, of course, that further research will whittle away at the edges of the imponderables and provide bases for sounder judgments that will reduce the imbalances and inefficiencies that may be present in our transportation system.

It bears repeating that the effectiveness of the great productive machine that has made possible such high standards of living in this country cannot be attributed alone to the wealth of our natural resources, the skill of our labor forces, and our efficient use of capital in mass production. It rests, also, in no small measure, upon the complex but remarkably capable transportation system that has been developed. And highway transportation, all things considered, is perhaps the most important of all. Not only is it a potent competitive force, but it complements all other transportation agencies and plays an indispensable role in the direct satisfaction of human wants. A continuation of the phenomenal progress of highway transportation of the past 50 years is one of the great challenges of the future.

Reaction of Aggregate with Low-Alkali Cement

(*Continued from page 56*)

As indicated by Davis, Stanton[5] made several references to the effect of small amounts of alkali–reactive aggregates on the expansion of mortar, and to the possibility of a relation between the amount of the alkali and the amount of the reactive aggregate. In his reply to comments regarding his paper in the 1942 Transactions of the American Society of Civil Engineers, Stanton states:

> Further attention is called to the low percentage of opal required for maximum results, as well as the remarkably low percentage (in some cases less than 0.5 percent) which produces excessive expansion in a relatively short time * * *.
>
> Test results available by July 1940, led to the conclusion that, since little, if any, measurable expansion was observed with deleterious particles below 80-mesh or 100-mesh size, the reason for the falling off in expansion as the percentage of such particles was increased beyond a reasonable percentage might be an accelerated reaction between the cement and finely dispersed mineral particles.
>
> Although the underlying cause may be that originally suspected, later tests showed that neutralization also occurs in the presence of excess percentages of coarser particles from which all finer particles have been removed. From this it would appear that, for maximum reaction, there is a relation between the amount of reactive mineral in any unit area and the alkali in the cement * * *.

In connection with a discussion of the effect of added sodium or potassium hydroxide to mortar or concrete, Stanton also states:

> It is probable that there is a different "pessimum" amount of alkali for various minerals, just as there is a different "pessimum" amount of different minerals for a given alkali content.

It is unfortunate that Stanton did not inquire further regarding the behavior of aggregates containing small amounts of reactive material or develop the relation between the amount of reactive mineral and the alkali in the cement more fully. Lack of definite information along these lines as well as incomplete knowledge of the behavior of low-alkali cements has caused the development of certain assumptions regarding high-alkali cements and highly reactive aggregates which may be entirely unwarranted.

Application of Findings

The application of these findings to the apparently anomalous results obtained in many previous investigations will readily be seen. Specific mention of only one of these investigations will be made. It will be recalled that in a study of the behavior of Platte River gravel as a concrete aggregate, Jackson and Kellermann[6] reported that the gravel was found to contain only 0.3 percent opal by weight. This small amount of opal was believed by many to be insufficient to cause distress of concrete, yet concrete pavements prepared with this gravel persistently developed multiple cracking and excessive expansion. From the information shown here, the use of an aggregate containing very small amounts of opal may be sufficient with a particular cement to cause distress of concrete due to internal expansion.

In determinations of the availability for chemical reaction of alkali in portland cement, it seems to be highly desirable to make tests of mortar using sand containing several different amounts of reactive mineral. The data given here show that determinations should be made of the quantity of reactive material which will give the maximum amount of expansion with the particular cement under test. Only by this method can a worth-while comparison of the chemical activity of different cements be obtained.

If concrete is prepared with a low-alkali cement, there is no surety that a nonexpansive material will be obtained unless the aggregate contains no reactive mineral or sufficient reactive mineral to give a low value for the alkali–reactive aggregate ratio. In the case of high-alkali cement, the addition of a small amount of reactive matter to the concrete to control the reaction may cause the development of more expansion than if the reactive matter were omitted. In either case, it is believed that tests should be made using the cement, the aggregates, and the admixture, if any, which are considered for use in the proposed construction. Only by the preparation and testing of concrete as it will be used can information of the stability of the material be obtained.

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[5] See footnote 2, p. 50.

[6] *Volume changes in sand-gravel concrete,* by F. H. Jackson and W. F. Kellermann. Proceedings of the Highway Research Board, vol. 22, 1942.

Reaction of Aggregate with Low-Alkali Cement

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by DONALD O. WOOLF
Senior Materials Engineer

Abnormal expansion of concrete, with resulting damage, has been widely observed for a number of years. Previous study of this phenomenon has disclosed the cause to lie in the reaction of alkali in the cement with certain reactive materials in the aggregate. It has generally been considered in the past that the amount of expansion was related to the alkali content of the cement—the larger the alkali content, the greater the expansion.

From the test data reported in this article, it now appears that excessive expansion of concrete is governed, not by the cement alkali content alone, but by the relation of the alkali content of the cement to the amount of reactive material in the aggregate. Low-alkali cement used with aggregate containing a small amount of reactive material may result in objectionable expansion, while the same cement with aggregate containing a relatively large amount of reactive material may have little if any reaction.

It is evident that, if excessive expansion of concrete is to be avoided, the alkali content of the cement and the amount of reactive material in the aggregate must both be investigated, and their reactive effect explored. Tests should be made using the cement, the aggregates, and the admixture, if any, which are considered for use in proposed construction. Only by the preparation and testing of concrete as it will be used can information on the stability of the material be obtained.

TESTS of 36 portland cements varying in alkali content [1] from 0.22 percent to 0.99 percent indicate, as reported in this article, that mortar prepared with any portland cement may develop excessive expansion due to the reaction between the aggregate and sodium oxide or potassium oxide, the alkalis in portland cement. An expansion of 0.1 percent or more at an age of 1 year or less is considered as an excessive amount of expansion. The quantity of alkali-reactive material in the sand needed to obtain a maximum reaction appears to be dependent on the amount of alkali in the cement. If the aggregate contains a relatively large amount of alkali-reactive material, a low-alkali cement probably will cause little if any reaction, but the same cement used with a sand having only a small amount of reactive material may cause an objectionable expansion. Definite information regarding the stability of concrete with respect to expansion caused by chemical reaction can be obtained only by tests of concrete containing the cement and the aggregates to be used, combined in the same proportions as specified for the work.

Previous Tests and Conclusions

In his first report [2] on the chemical reaction between cement and aggregates, T. E. Stanton concluded:

> The chemical reaction producing excessive expansion apparently occurs only when the portland cement component contains an appreciable percentage of alkali in the form of sodium and potassium oxides. It is of an intensity proportional to the percentage of such oxides, apparently being of such low order as to be negligible when the alkali content is less than 0.6 percent.

Stanton's use of 0.6 percent alkali to separate high-alkali from low-alkali cements has been accepted widely and the procedure developed by the laboratory of the California Division of Highways for the determination of reactive aggregates has been used extensively. Under this procedure comparisons are made between the expansion developed by a mortar containing a high-alkali cement (0.6 percent or more) in combination with the aggregate under investigation and the expansion of similar mortars containing the same cement and one or more standard aggregates of known behavior. These standard aggregates include a nonreactive sand (silica sand from Ottawa, Ill., is used by the Bureau of Public Roads), and reactive sands which are obtained by the addition to the nonreactive sand of small amounts of material containing opal. Several opal-bearing limestones and cherts were tested by the Bureau of Public Roads but different shipments of these materials were found to vary in their reactivity with alkali and the use of practically pure opal was adopted.

Data published by Stanton and data subsequently obtained by the Bureau of Public Roads showed that the expansion of portland cement mortar was influenced by the amount and grain size of the reactive material in the sand as well as by the particular cement used. In figure 1, the effect on the expansion of mortar of different percentages of reactive material in the aggregates is shown.[3] Information developed in the laboratory of the Bureau of Public Roads regarding the effect of size of grain of reactive limestone on the expansion of mortar is given in figure 2.

Stanton's tests (fig. 1) indicate that, in the case of chert, maximum expansion was developed when approximately 5 percent of the reactive material was used. On the other hand, when the reactive limestone was used, Stanton's tests showed that 20 percent was required to develop maximum expansion as compared with 10 percent in the Bureau's tests (fig. 2). As the result of this work, the Bureau of Public Roads adopted the practice of using, as a standard reactive aggregate, Ottawa silica sand to which was added 5 or 10 percent crushed opal by weight. At first, little consideration was given to the size of the crushed opal other than it pass the No. 8 sieve and be retained on the No. 50 sieve. Later, it was found that variations in the size of the opal caused nonuniform results from test to test, and the practice of using a single size of crushed opal was adopted. For use with Ottawa sand, the opal was sieved to the No. 16 to No. 30 size.

Failures with Low-Alkali Cement

In the earlier phases of investigations of the alkali–aggregate reaction, consideration was given almost entirely to determining if a particular aggregate could be used safely with high-alkali cement. Test specimens were prepared with the aggregate in question and

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[1] Sodium oxide and potassium oxide are reported here as "total alkali" with the percentage of alkali given as the sum of the sodium oxide content plus 0.658 of the potassium oxide content.

[2] *Expansion of concrete through reaction between cement and aggregate* by Thomas E. Stanton. Transactions of the American Society of Civil Engineers, vol. 107, pp. 54-81 1942.

[3] Taken from figure 2, *Durability of Concrete as Affected by Aggregates*, by Thomas E. Stanton. National Sand and Gravel Association Circular No. 28, 1947.

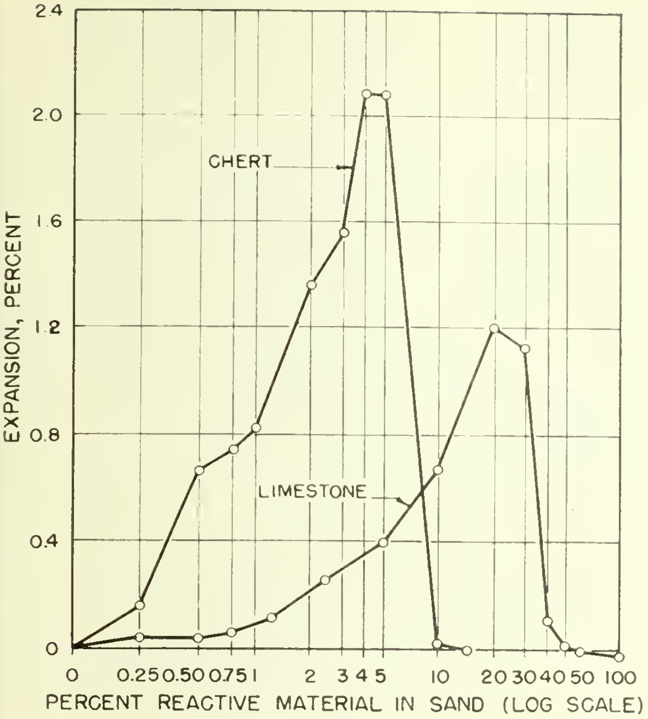

Figure 1.—Effect of amount and kind of reactive material on expansion of 1: 2 mortar at age of 48 months. (Reactive material substituted for equivalent percentages of neutral sand; alkali in cement, 1.14 percent; mortar bars stored in sealed containers at 70° F. After Stanton.)

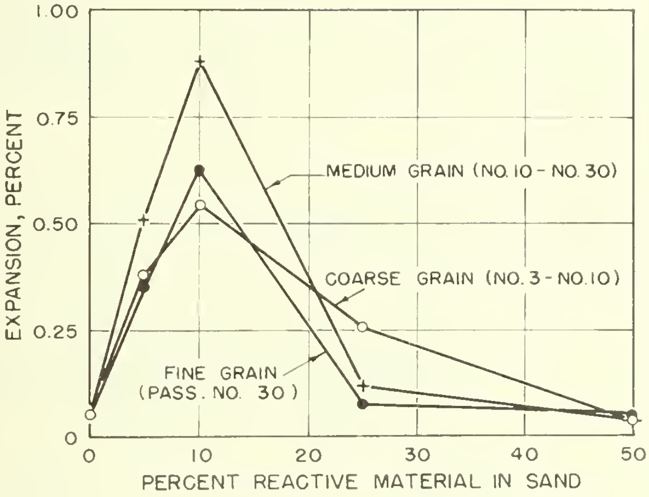

Figure 2.—Effect of amount and size of grain of reactive material on expansion of 1: 2 mortar at age of 36 months. (Reactive siliceous magnesium limestone substituted for equal percentages of Potomac River sand; storage in moist air at 130° F.)

each of several high-alkali cements. Occasionally a low-alkali cement was also used, but the expansion of low-alkali cement mortar was usually so small that it was believed a waste of effort to test such material. When interest developed in possible differences in the activity of different high-alkali cements, the same belief of the nonreactivity of low-alkali cements prevailed and, except for an occasional token test, little thought was given to studies of low-alkali cements.

With broadening of knowledge of the occurrence of alkali–aggregate reaction in the field, reports were received of the failure of concrete containing low-alkali cement. In some instances the type of failure was reported to resemble that found when reactive aggregates were used with high-alkali cement. The receipt of this information recalled certain anomalies found in laboratory work with low-alkali cements in which excessive amounts of expansion of mortar had been observed. A review of the test records showed many instances where mortar prepared with low-alkali cement developed excessive expansion; in other work, mortars containing high-alkali cement and reactive aggregates gave only small amounts of expansion. The number of these results as well as several instances of duplication of materials used and duplication of results, indicated that possibly some features of the alkali–aggregate reaction had not been given the attention they deserved.

Available Data Restudied

These observations indicated the desirability of investigating more fully the behavior of the low-alkali cements in combination with sands containing small amounts of reactive material. Instead of initiating a new research project, however, it was decided to study certain available data from tests which had been under way for various periods up to 18 months. These tests involved determination of the expansion of mortar bars made with a large number of cements of varying alkali content in combination with aggregates containing very small percentages of opal.

A review of the data for this group of tests showed that 36 cements from 35 different mills had been tested with graded Ottawa sand containing 1, 2, and 5 percent of opal. Chemical analyses of these cements are given in table 1. Fifteen of the cements had an alkali content of less than 0.60 percent, and nine contained less than 0.50 percent alkali. Only one round of test specimens was made with ten of the cements, but two to five rounds, depending on the amount of material available, were made on different days with the other cements. Each round consisted of two specimens prepared with each of the three test sands. A water-cement ratio of 0.5 by weight was used for each mortar.

All specimens were stored in moist air at a temperature of 100° F., and determinations of change in volume were made at periodic intervals. Over half of the specimens were kept under test to an age of 18 months; tests of specimens prepared with the other cements were discontinued at earlier ages when an excessive amount of expansion was found. It is unfortunate that all specimens were not tested to an age of at least a year but the overcrowding of the storage facilities required the removal of some of the specimens. In the accompanying data showing expansion of mortar, the number of specimens used to obtain the values reported is shown. In a number of cases, breakage of a test specimen or discontinuance of tests for another reason caused the deletion of all test determinations for the specimens so affected.

Surprising Results Obtained

The results of tests for expansion of mortar are given in table 2, and are shown graphically in figures 3A and 3B. The results obtained show a surprising trend. All of the cements show more expansion when used with sand containing either 1 or 2 percent of opal than when used with sand containing 5 percent of opal. It has been known for some time that if sufficient opal were added to inert sand, a mortar could be obtained which would have little expansion. However, the amount of opal needed to obtain this has been believed to be 25 percent or more, and sand containing only 5 percent of opal was considered to be highly reactive. To find that smaller amounts of opal in a sand would cause greater amounts of expansion raises a question as to the validity of the results obtained in earlier investigations, particularly those concerned with the availability for reaction of alkali in cement.

At least one of the three mortars prepared with each cement expanded 0.10 percent or more at an age of 1 year or less. An expansion of this amount at an age of 1 year has recently been included in a proposed revision of the standard specifications for concrete aggregates, American Society for Testing Materials designation C 33, to indicate an aggregate which is objectionably reactive with alkali in cement. On the basis of this maximum permissible expansion, the results of these tests may be interpreted to show that

Table 1.—Chemical analysis and compound composition of cements (in percent)

Cement No.	Chemical analysis								Total alkali	Compound composition		
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Na_2O	K_2O		C_3S	C_2S	C_3A
1	20.3	6.5	2.2	67.3	0.8	1.8	0.04	0.28	0.22	68	7	14
2	20.4	6.5	2.8	63.7	3.0	2.1	.14	.13	.23	50	21	12
3	22.1	5.9	2.0	65.6	1.0	1.9	.39	.08	.24	51	25	12
4	21.7	6.0	2.2	65.5	1.0	1.8	.09	.28	.27	53	22	12
5	21.3	5.9	2.9	65.3	1.2	2.3	.07	.34	.29	54	21	11
6	20.9	6.2	2.1	66.1	1.0	1.8	.08	.32	.29	60	14	14
7	19.7	7.1	3.2	65.1	1.3	2.1	.08	.42	.36	57	14	13
8	22.1	5.1	1.8	66.6	1.1	1.7	.32	.11	.39	61	17	10
9	20.5	6.2	3.2	65.2	.9	2.3	.15	.43	.43	57	16	11
10	----	-----	-----	----	----	----	.20	.47	.51	----	----	----
11	19.7	6.7	3.5	65.1	.9	2.1	.14	.59	.53	59	12	12
12	21.4	5.8	2.4	65.6	1.0	1.9	.15	.59	.54	57	19	11
13	20.7	5.7	2.6	63.6	3.2	1.6	.19	.53	.54	55	18	11
14	22.0	5.6	3.0	65.0	.6	1.8	.26	.48	.58	50	25	10
15	21.4	5.4	2.5	66.2	.9	1.9	.44	.23	.59	62	15	10
16	21.8	5.1	2.8	64.7	1.5	1.8	.21	.60	.60	54	22	9
17	----	-----	-----	----	----	----	.33	.43	.61	----	----	----
18	20.9	6.4	1.9	63.6	3.0	2.2	.25	.56	.62	48	24	14
19	22.6	5.5	3.0	64.0	.8	1.9	.17	.70	.63	42	33	10
20	----	-----	-----	----	----	----	.35	.43	.63	----	----	----
21	----	-----	-----	----	----	----	.36	.44	.65	----	----	----
22	20.5	6.1	3.2	64.6	1.1	2.2	.45	.34	.67	55	17	11
23	----	-----	-----	----	----	----	.31	.54	.67	----	----	----
24	----	-----	-----	----	----	----	.27	.62	.68	----	----	----
25	21.4	5.0	3.6	63.4	2.8	1.6	.62	.10	.69	52	22	7
26	----	-----	-----	----	----	----	.32	.60	.71	----	----	----
27	21.1	4.4	4.0	63.4	3.1	1.7	.23	.74	.72	58	17	5
28	----	-----	-----	----	----	----	.34	.61	.74	----	----	----
29	20.9	6.2	3.2	63.1	1.8	1.8	.15	.67	.77	47	25	11
30	22.2	5.4	2.6	62.5	3.3	1.8	.29	.77	.80	41	33	10
31	20.1	6.6	2.8	63.4	2.9	1.7	.40	.61	.80	52	18	13
32	21.7	4.9	3.1	63.6	2.3	1.8	.14	1.04	.82	51	24	8
33	----	-----	-----	----	----	----	.34	.76	.84	----	----	----
34	----	-----	-----	----	----	----	.34	.90	.93	----	----	----
35	21.4	5.3	2.6	64.9	1.4	1.5	.95	.06	.99	58	18	10
36	22.6	6.2	1.8	62.1	1.7	2.2	.73	.39	.99	30	42	14

all cements, even with as little as 0.22 percent alkali, may furnish mortar which will expand excessively if sand with a critical amount of alkali-reactive substance is used.

All of the eight cements in the group which had less than 0.40 percent alkali (cements 1–8) gave more expansion when used with sand containing 1 percent opal than when used with sand containing larger amounts of the reactive mineral. Twenty-five of the twenty-eight cements having more than 0.40 percent alkali (cements 9–36) gave as much, or more, expansion with sand containing 2 percent opal as with sand containing 1 percent and all showed more expansion with 2 percent opal than with 5 percent.

The relation between the alkali content of the cement and the expansion of mortar at ages of 1 and 18 months is shown in figure 4. Mortar prepared with sand containing 1 percent opal developed maximum expansion at 18 months with cement containing 0.6 to 0.7 percent alkali. With greater amounts of alkali, a regression of the expansion of the mortar was found.

When sand with 2 percent opal was used, maximum expansion of mortar at 18 months was obtained with cement containing about 0.9 percent alkali. The data shown for mortar prepared with sand having 5 percent opal indicate that for this condition the maximum expansion of mortar at 18 months may be obtained with cement having considerably more alkali than any cement used in these tests. It seems probable that this curve would flatten out as do those for 1 and 2 percent opal.

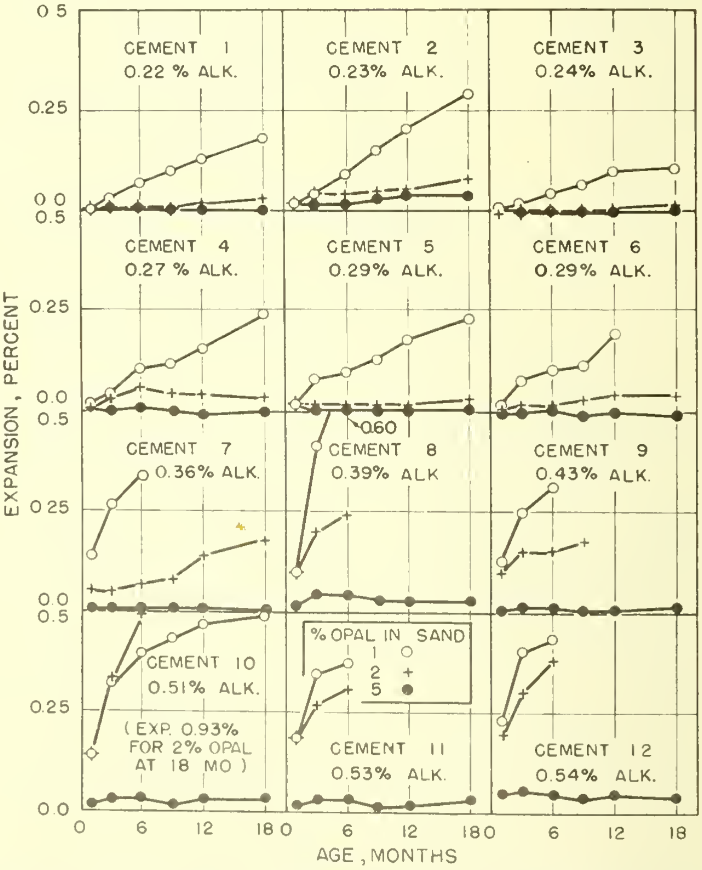

Figure 3A.—Expansion of 1: 2 mortar prepared with cements having various percentages of alkali content and sand containing 1, 2, and 5 percent opal.

Expansion at Eighteen Months

The average curves shown separately for 1, 2, and 5 percent opal are replotted in the summary chart in figure 4 for the purpose of ready comparison. As both sets of curves in this chart show similar trends and as the tests at 18 months may be considered to represent more closely the ultimate behavior of these mortars, reference to the curves for the tests at 18 months will be made. Possibly the most important feature shown is that when used with cement of low-alkali content, 0.4 percent or less, sand containing a small amount of opal will produce mortar having an excessive amount of expansion (over 0.1 percent) while sand with a greater opal content may furnish mortar having but little expansion. When cements with less than about 0.4 percent alkali were used, mortars prepared with sand containing 1 percent opal had greater expansion than those prepared with the other two sands. The curve for mortar prepared with sand containing 2 percent opal shows low amounts of expansion for cements with an alkali content of 0.3 percent or less, and that for sand with 5 percent opal shows expansions of less than 0.1 percent for cements containing about 0.6 percent alkali or less.

Sand containing 1 percent opal furnished mortar with a greater amount of expansion than sand containing 5 percent opal when used with cement containing about 0.9 percent

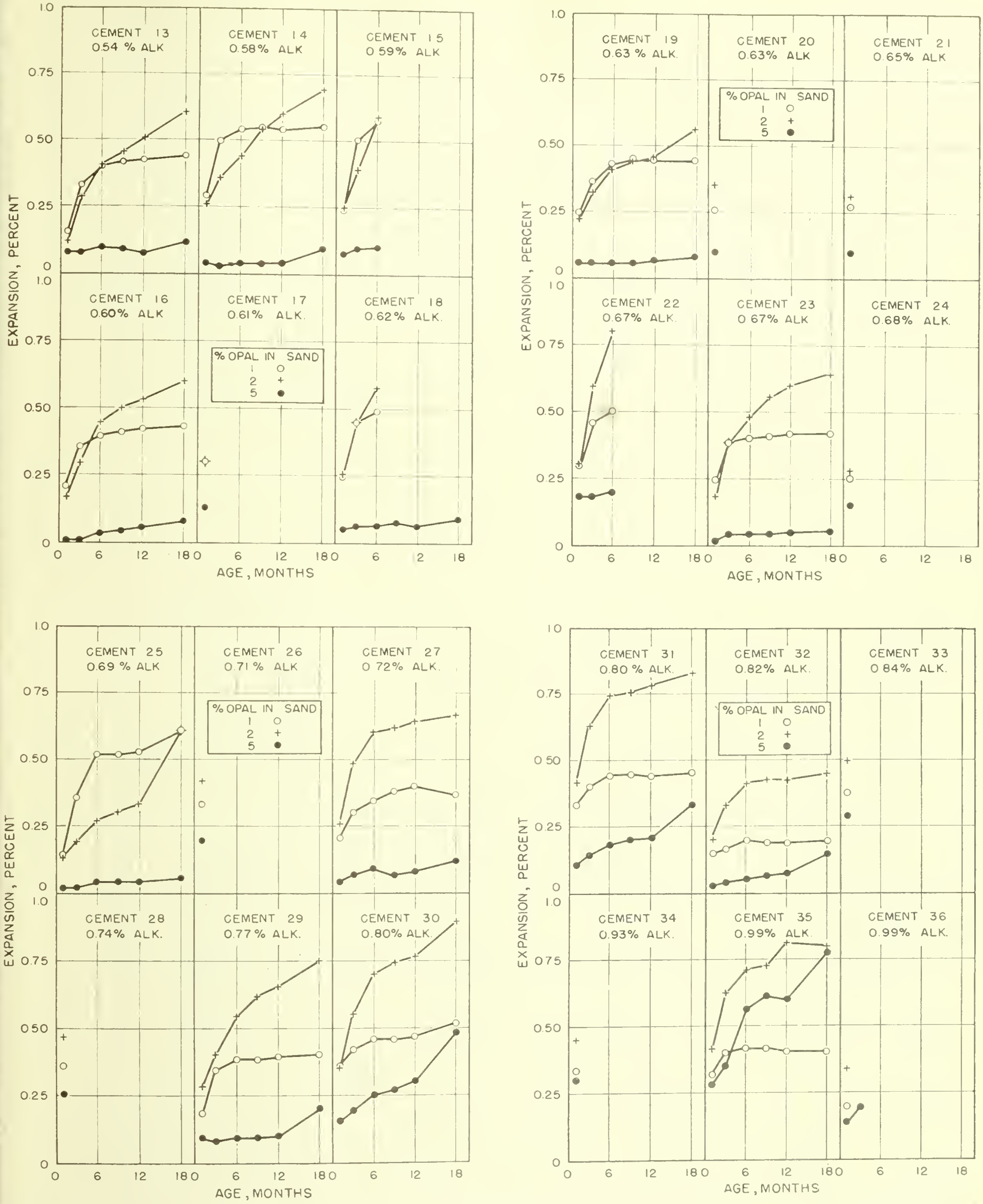

*Figure 3B.—Expansion of 1: 2 mortar prepared with cements having various percentages of alkali content and sand containing 1, 2, and 5 percent opal—*Continued.

Table 2.—Expansion of 1:2 mortar prepared with Ottawa sand containing different amounts of opal (moist air storage at 100° F.)

Cement number	Alkali content of cement	Opal content of sand	Number of specimens tested	Expansion at age of— 1 month	3 months	6 months	9 months	12 months	18 months
	Percent	*Percent*		*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	0.22	1	6	0.00	0.03	0.07	0.10	0.13	0.18
		2	6	.00	.01	01	.01	.02	.03
		5	6	.00	.01	.01	.00	.00	.00
2	.23	1	2	.02	.04	.09	.16	21	.29
		2	2	.02	.04	.04	.05	.06	.08
		5	2	.02	.02	.02	.03	.04	.04
3	.24	1	6	.01	.02	.04	.07	.10	.11
		2	6	.00	.01	.00	.00	.01	.02
		5	6	.00	.00	.00	.00	.00	.00
4	.27	1	4	.02	.04	.11	.12	.16	.24
		2	4	.01	.03	.06	.04	.04	.03
		5	6	.01	.00	.01	.00	−.01	.00
5	.29	1	8	.02	.08	.10	.13	.18	.23
		2	8	.02	.02	.02	.02	.02	.03
		5	8	.02	.01	.01	.01	.01	.01
6	.29	1	4	.02	.08	.11	.12	.19	------
		2	6	.01	.02	.02	.03	.04	.04
		5	6	.00	.00	.01	−.01	.00	−.01
7	.36	1	6	.14	.27	.34	------	------	------
		2	6	.05	05	.07	.08	.14	.18
		5	6	.01	.01	.01	.01	.01	.00
8	.39	1	8	.10	.42	.60	------	------	------
		2	8	.10	.20	.24	------	------	------
		5	8	.02	.04	.04	.03	.03	.03
9	.43	1	6	.13	.25	.32	------	------	------
		2	4	.10	.16	.16	.18	------	------
		5	6	.01	.02	.02	.01	.01	.02
10	.51	1	2	.14	.32	.39	.43	.47	.49
		2	2	.14	.33	.49	.66	.77	.93
		5	2	.02	.03	.03	.02	.03	.03
11	.53	1	4	.18	.34	.37	------	------	------
		2	4	.18	.27	.31	------	------	------
		5	6	.02	.03	.03	.02	.02	.03
12	.54	1	4	.23	.40	.43	------	------	------
		2	4	.19	.30	.38	------	------	------
		5	6	.05	.06	.05	.04	.05	.04
13	.54	1	2	.16	.33	.40	.42	.43	.44
		2	2	.12	.28	.41	.46	.51	.61
		5	2	.08	.08	.10	.09	.08	.12
14	.58	1	2	.29	.50	.54	.55	.54	.56
		2	2	.26	.36	.44	.54	.60	.69
		5	2	.04	.03	.04	.04	.04	.10
15	.59	1	6	.24	.51	.58	------	------	------
		2	6	.25	.39	.59	------	------	------
		5	6	.08	.10	.11	------	------	------
16	.60	1	10	.21	.35	.39	.41	.42	.43
		2	10	.17	.29	.44	.50	.53	.60
		5	10	.01	.01	.03	.04	.06	.08
17	.61	1	6	.30	------	------	------	------	------
		2	6	.30	------	------	------	------	------
		5	6	.13	------	------	------	------	------
18	.62	1	6	.25	.45	.49	------	------	------
		2	6	.26	.45	.58	------	------	------
		5	6	.06	.07	.07	.08	.07	.09
19	0.63	1	2	0.24	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		2	2	.22	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		5	2	.06	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
20	.63	1	6	.26					
		2	6	.35	------				
		5	6	.10					
21	.65	1	6	.27					
		2	6	.31					
		5	6	.10					
22	.67	1	4	.29	.46	[illegible]			
		2	4	.30	.59	[illegible]			
		5	4	.18	.18	[illegible]			
23	.67	1	2	.24	.38	[illegible]	[illegible]	[illegible]	[illegible]
		2	2	.18	.38	.48	[illegible]	[illegible]	.64
		5	2	.02	.04	.01	[illegible]	[illegible]	[illegible]
24	.68	1	6	.25					
		2	6	.28			------		
		5	6	.15					
25	.69	1	2	.14	.36	.52	.52	.53	.61
		2	2	.13	.19	.27	.30	[illegible]	.61
		5	2	.02	.02	.04	.04	[illegible]	.06
26	.71	1	6	.33	------				
		2	6	.42	------				
		5	6	.19	------				
27	.72	1	2	.21	.30	.34	.38	.40	[illegible]
		2	2	.26	.48	.60	.62	.64	[illegible]
		5	2	.04	.07	.09	.07	.08	.12
28	.74	1	8	.36	------			------	
		2	8	.47	------			------	
		5	8	.26	------	------		------	
29	.77	1	2	.18	.34	.38	.38	.39	.40
		2	2	.28	.40	.54	.62	.66	.75
		5	2	.09	.08	.09	.09	.10	.20
30	.80	1	10	.36	.42	.46	.46	.47	.52
		2	10	.35	.55	.70	.74	.77	.90
		5	10	.16	.19	.25	.27	30	.48
31	.80	1	10	.33	.40	.41	.45	.44	.46
		2	10	.42	.63	.74	.76	.78	.83
		5	10	.11	.14	.18	.20	.21	.33
32	.82	1	10	.15	.17	.20	.19	.19	.20
		2	10	.21	.33	.42	.43	.43	.46
		5	10	.03	.04	.06	.07	.08	.15
33	.84	1	6	.38	------	------	------	------	------
		2	6	.50	------	------	------	------	------
		5	6	.29	------	------	------	------	------
34	.93	1	6	.33	------	------	------	------	------
		2	6	.45	------	------	------	------	------
		5	6	.30	------	------	------	------	------
35	.99	1	10	.32	.40	.42	.42	.41	.41
		2	10	.42	.63	.72	.73	.82	.81
		5	10	.28	.35	.57	.62	.61	78
36	.99	1	2	.20	------	------	------	------	------
		2	2	.34	------	------	------	------	------
		5	2	.14	.20	------	------	------	------

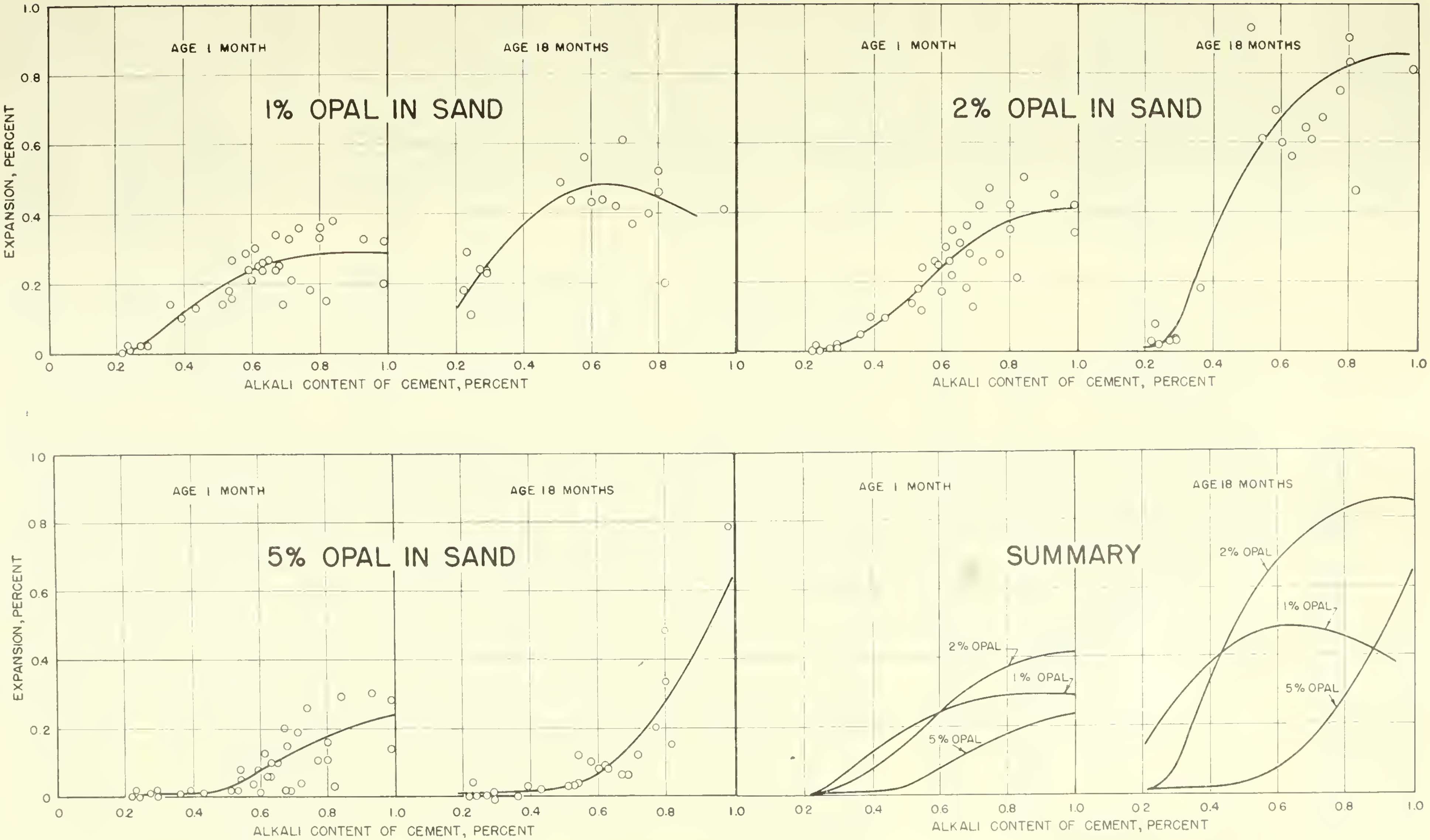

Figure 4.—Relation between alkali content of cement and expansion of 1: 2 mortar prepared with sand containing 1, 2, and 5 percent opal.

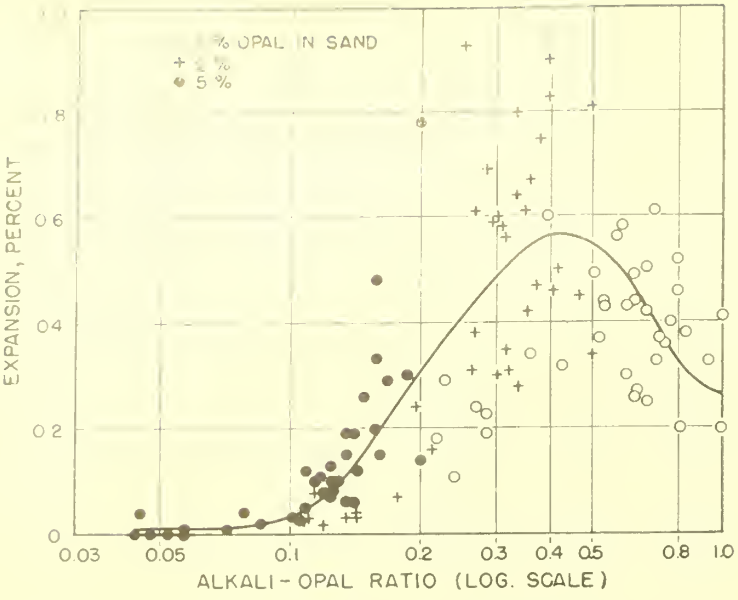

Figure 5.—Relation between alkali–opal ratio and expansion of 1 : 2 mortar prepared with sand containing 1, 2, and 5 percent opal.

alkali or less, and sand containing 2 percent opal furnished mortar which expanded more than that prepared with 5 percent opal for all cements included in these tests. The trend of the curves indicates that for cements containing about 1.1 percent alkali or more, mortar prepared with sand containing 5 percent opal would have greater expansion than that prepared with sand containing 2 percent opal.

Alkali–Opal Ratio Important

From the above it appears that, for mortar of maximum expansion, the alkali content of the cement must increase as the opal content of the sand increases within the range of chemical content of the materials used in these tests. This suggests that a ratio of alkali content to opal content might be used to indicate for mortar of given proportions the combination of cement and aggregate which will furnish the most undesirable characteristics to the mortar, and also the combinations which will produce mortar having relatively a small amount of expansion. The use of such a ratio is shown in figure 5.

In this figure, the values plotted are those for each of the three mortars prepared with a given cement at the greatest age shown in table 2 common for all three mortars. Some of these values were determined at an age of 1 month, others at ages of 6, 12, and 18 months. Although the values plotted have considerable dispersion, they serve to illustrate the development of maximum expansion for a particular combination of cement and reactive sand and not necessarily for cement of the maximum alkali content.

With the materials used in these tests, the maximum amount of expansion of mortar was obtained when the alkali–opal ratio was approximately 0.4. This is, of course, an average value representing the three reactive sands used in these tests. Low amounts of expansion, that is expansions of less than the 0.1 percent cited by the proposed A.S.T.M. specification C 33, were obtained with combinations of cement and aggregate having an alkali–opal ratio of 0.13 or less. Thus it appears that highly reactive aggregates and cements containing high amounts of alkali may furnish mortar or concrete with less expansion than aggregates containing small amounts of reactive material and low- or moderate-alkali cements.

Maximum Expansion Considered

Frequent mention has been made here of the maximum expansion of mortar or concrete. In consideration of the expansion of materials due to the alkali-aggregate reaction, the development of an objectionable amount of expansion, which may be much smaller than the maximum amount, must not be ignored. However, in comparisons between different materials, the maximum expansion obtained may be that considered in the selection of a material or a combination of materials for use. In this discussion, consideration of the maximum expansion found is believed permissible provided attention is directed to the desirability of establishing, when needed, limiting values for the expansion.

It is probable that the alkali-reactive aggregate ratio for maximum expansion of mortar or concrete will vary depending on the characteristics of the reactive aggregate used. If the reactive aggregate is wholly opal, the ratio for maximum expansion may be close to that determined here. If the reactive aggregate is semi-opal, or volcanic glass, or any of other alkali-soluble forms of silica, the ratio which will give the most undesirable properties to the cement product may be quite different from that mentioned here.

Many additional tests would be required to develop this relation fully, and at present there seems to be little need for this complete determination. The number of cements included in these tests and the uniformity of the results obtained appear to be sufficient to warrant the statement that, for the maximum expansion of a mortar of given proportions, the controlling factor is the alkali–reactive aggregate ratio and not the total alkali in the cement.

Particular attention was given to the magnesia content of each cement, as it has been stated that magnesia joins with the alkali in promoting reaction with the aggregate. Only five of the cements for which complete analyses are shown have 3.0 percent or more magnesia, and none exceed the specification limit of 5.0 percent. Only one of these cements (cement 2) produced mortar with a significantly greater amount of expansion than that shown by mortars prepared with cements of about the same alkali content but with appreciably less magnesia. No well-defined relation between any of the determinations of the oxides, other than the alkali, and the expansion of mortar can be found in these data.

Similar Results Found by Others

Subsequent to the development of the conclusions reported here, attention was drawn to work along similar lines by C. E. S. Davis of the Commonwealth Scientific and Industrial Research Organization of Australia.[4] Tests were made of mortar prepared with inert sand containing 2.5, 5.0, and 10.0 percent opal and cements having from 0.03 to 1.99 percent sodium oxide. The cements were prepared in the laboratory. In a summary of the results of these tests, the following statements were made:

> The rate and amount of expansion depended also on the opal content of aggregate and on the opal to soda ratio. The most striking illustration of this was that low-soda clinkers (containing 0.35–0.49 percent Na_2O), which did not cause significant expansion with aggregates containing 5 percent opaline rock, eventually expanded mortar made with aggregates containing 2.5 percent of opaline rock. It may thus be inferred that clinkers of even lower soda content may cause expansion if made into mortar with aggregates containing 2.5 percent or less of opaline rock.
>
> As the soda content of clinker increased, the amount of opal required for maximum expansion (Stanton's "pessimum proportion") also increased.

Although he did not develop the information as fully as might be desirable, Davis' reference to the opal to soda ratio and his inference regarding the behavior of cements of very low-alkali content agree substantially with the findings of this investigation.

(*Continued on page 49.*)

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[4] *The effect of soda content and of cooling rate of portland cement clinker on its reactions with opal in mortar,* by C. E. S. Davis. Australian Journal of Applied Science, vol. 2, No. 1, March 1951.

U. S. GOVERNMENT PRINTING OFFICE: 1952

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1952

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$6,031	$35,523	$17,882	553.3	$8,819	$4,828	192.2	$21,750	$10,764	428.5	$66,092	$33,474	1,174.0
Arizona	822	4,120	2,748	107.5	1,002	524	20.3	9,719	6,562	128.2	14,841	9,834	256.0
Arkansas	3,172	10,278	5,461	285.9	7,575	3,752	213.8	17,135	8,704	494.7	34,988	17,917	994.4
California	3,197	20,846	6,271	147.7	19,355	9,451	72.9	86,968	42,264	252.7	127,169	57,986	473.3
Colorado	4,643	5,240	2,924	116.7	2,547	1,397	64.4	13,988	7,125	280.8	21,775	11,446	461.9
Connecticut	6,425	2,104	1,102	5.8	2,161	1,055	9.0	10,546	5,579	16.4	14,811	7,736	31.2
Delaware	2,317	335	165	1.7	2,023	1,011	8.0	6,063	2,670	49.9	8,421	3,846	59.6
Florida	3,050	17,854	9,187	305.2	7,792	3,976	129.5	14,988	7,599	270.8	40,634	20,762	705.5
Georgia	3,630	19,168	9,898	487.2	9,605	4,805	111.2	34,694	16,500	582.1	63,467	31,203	1,180.5
Idaho	5,416	8,220	5,122	228.5	3,385	2,181	45.3	6,672	4,299	160.2	18,277	11,602	434.0
Illinois	10,794	51,148	28,253	461.0	17,988	9,017	205.8	82,454	41,575	715.7	151,590	78,845	1,382.5
Indiana	7,347	40,169	20,343	259.4	7,764	4,445	162.0	28,861	15,345	333.7	76,794	40,133	755.1
Iowa	1,932	12,870	6,868	358.8	10,677	5,429	448.3	19,841	9,874	885.5	43,388	22,171	1,692.6
Kansas	7,179	10,784	5,239	1,017.4	5,349	2,691	524.8	16,670	8,691	563.1	32,803	16,621	2,105.3
Kentucky	1,571	21,157	11,257	416.0	7,120	3,483	135.0	16,513	8,583	327.3	44,790	23,323	878.3
Louisiana	3,031	15,455	7,381	114.4	10,933	4,986	31.5	20,621	10,289	216.5	47,009	22,656	362.4
Maine	823	10,764	5,622	100.3	1,111	677	7.5	9,705	4,878	74.5	21,580	11,177	182.3
Maryland	4,956	9,919	4,558	71.4	1,700	851	68.4	10,186	5,468	19.1	21,805	10,877	158.9
Massachusetts	3,855	4,861	2,559	18.1	9,444	4,486	8.4	47,965	23,570	47.4	62,270	30,615	73.9
Michigan	2,958	29,833	15,046	490.8	11,719	5,858	240.8	59,152	26,036	350.9	100,704	46,940	1,082.5
Minnesota	3,749	9,612	5,144	1,126.5	9,340	4,674	723.0	27,828	15,113	999.2	46,780	24,931	2,848.7
Mississippi	4,063	17,330	8,921	559.7	5,884	2,908	188.0	16,499	8,628	468.4	39,713	20,457	1,216.1
Missouri	9,370	27,714	14,084	896.5	15,011	7,505	252.9	40,845	21,422	659.5	83,570	43,011	1,808.9
Montana	8,709	9,307	5,519	292.7	4,669	2,721	71.1	20,719	12,475	352.2	34,695	20,715	716.0
Nebraska	11,196	12,027	6,311	583.1	5,937	2,948	156.0	18,826	9,334	601.6	36,790	18,593	1,340.7
Nevada	2,519	7,338	5,831	250.7	175	146	35.9	5,653	4,704	222.4	13,166	10,681	509.0
New Hampshire	1,592	5,416	2,847	28.7	615	311	5.2	6,898	3,438	46.9	12,929	6,596	80.8
New Jersey	1,645	10,336	5,086	46.1	8,506	4,254	14.4	30,967	15,316	25.4	49,809	24,656	85.9
New Mexico	1,371	4,616	2,954	150.9	2,827	1,788	104.9	13,240	8,458	306.6	20,683	13,200	562.4
New York	24,122	96,297	50,015	205.2	31,993	14,989	177.9	107,540	49,118	399.4	235,830	114,122	782.5
North Carolina	4,403	24,580	11,967	437.7	5,928	2,831	142.7	23,715	11,620	464.9	54,223	26,418	1,045.3
North Dakota	1,551	7,786	4,025	1,116.8	8,679	4,431	783.5	9,901	4,971	796.4	26,366	13,427	2,696.7
Ohio	8,930	36,801	18,229	127.0	16,591	8,524	58.4	76,994	38,903	246.3	130,386	65,656	431.7
Oklahoma	2,659	15,944	8,822	224.9	7,398	3,884	169.3	23,449	12,407	263.4	46,791	25,113	657.6
Oregon	1,708	2,418	1,429	27.1	3,327	1,961	85.1	17,957	10,242	238.9	23,702	13,632	351.1
Pennsylvania	5,765	38,222	19,103	122.5	24,397	12,160	43.7	73,733	36,609	198.9	136,352	67,872	365.1
Rhode Island	771	4,355	2,178	35.0	2,380	1,190	8.2	17,212	8,984	24.9	23,947	12,352	68.1
South Carolina	3,516	10,400	5,643	327.3	3,915	1,945	217.4	15,867	8,126	362.3	30,182	15,714	907.0
South Dakota	1,205	7,704	4,441	662.0	4,186	2,440	246.6	13,897	8,046	774.1	25,787	14,927	1,682.7
Tennessee	3,745	12,178	5,803	408.3	13,581	6,501	351.2	30,336	14,437	444.2	56,095	26,741	1,203.7
Texas	12,184	6,832	3,387	163.1	16,572	9,400	461.2	59,219	30,666	1,071.8	82,623	43,453	1,696.1
Utah	1,344	5,050	3,859	160.6	1,900	1,526	10.2	8,451	6,237	152.9	15,401	11,622	323.7
Vermont	1,132	4,903	2,636	46.1	623	321	8.6	6,870	3,441	56.5	12,396	6,398	111.2
Virginia	4,996	20,424	10,195	326.5	6,781	3,399	131.3	25,390	12,723	386.8	52,595	26,317	844.6
Washington	2,315	12,343	5,920	166.1	3,017	1,581	88.2	18,166	9,106	174.9	33,526	16,607	429.2
West Virginia	2,920	12,957	6,539	104.5	4,494	2,271	48.5	14,195	7,075	135.3	31,646	15,885	288.3
Wisconsin	4,198	22,637	12,014	436.5	9,757	4,445	223.8	31,276	15,893	474.4	63,670	32,352	1,134.7
Wyoming	266	3,224	2,109	99.0	1,656	1,098	63.7	9,306	6,137	159.8	14,186	9,344	322.5
Hawaii	1,542	5,047	2,371	8.3	2,495	1,235	10.7	11,092	4,489	21.3	18,634	8,095	40.3
District of Columbia	618	13,459	5,909	1.4	2,654	1,324	.5	3,028	1,707		19,141	8,940	1.9
Puerto Rico	5,145	6,467	3,068	39.2	3,842	1,845	18.7	10,754	5,080	31.2	21,063	9,993	89.1
TOTAL	222,398	804,372	414,245	14,727.1	377,199	191,459	7,599.9	1,324,314	671,280	16,758.8	2,505,885	1,276,984	39,085.8

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OCTOBER 1952

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

Recent Trends in Highway Bond Financing

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH AND THE RESEARCH REPORTS BRANCHES, BUREAU OF PUBLIC ROADS

Reported by HUGO C. DUZAN,
WILLIAM R. McCALLUM,
and THOMAS R. TODD,
Transportation Economists

A review of the principles of public credit as applied to highways indicates that bond-issue financing can be used advantageously to accelerate the road-improvement program, particularly in the construction of expressways, large structures, and other primary improvements. The interest cost of a bond issue is justified by the advantages derived from the use of funds for construction now rather than at a future time as they accumulate from normal revenues. These advantages are derived in the form of added savings accruing to highway users because of earlier completion of the improvements and in the elimination of the cost of stop-gap improvements that would be necessary under a long-term current-revenue program.

The postwar need for highway modernization has caused an increasing trend among the States toward the use of credit financing. During the 5-year period 1946–50 the States, including special State authorities and commissions, issued $1,057 million in highway bonds (not including refunding issues), the counties and other local rural units issued $444 million, and the urban places $685 million, making a total for the period of $2,186 million. The amount of all highway and street debt outstanding at the end of 1950 was approximately $4.5 billion. The latest available figures for 1951 indicate that approximately $460 million in State highway issues alone were sold in that year.

Among bond issues at the State level the most publicized of recent developments has been the use of revenue-bond financing in the construction of toll roads. To a total of $54 million in toll-road bonds outstanding at the end of 1945, $449 million were added during the 6-year period 1946–51; $12 million were retired, leaving $491 million outstanding at the end of the period. Toll-bridge bonds increased in amount outstanding from $315 million to $445 million between 1945 and 1951.

Toll-free State issues outstanding at the end of 1945 were $1,269 million; $781 million were issued during the 6-year period and $539 million were redeemed, leaving $1,511 million outstanding at the end of 1951. Among toll-free issues there was a notable increase in the use of limited-obligation bonds, secured by a pledge of the proceeds of road-user taxes, the amount outstanding increasing from $97 million to $320 million during the 6-year period.

The study of individual issues in numerous States discloses wide variations in method and a tendency to experiment with different forms of credit financing. The toll-road movement continues vigorous; but two States, New Hampshire and New York, have taken steps to avoid the high debt-service charges associated with toll-revenue bonds by the use of general-obligation financing. Limitations on debt and the difficulties of amending State constitutions have been avoided in some States, notably Florida and Pennsylvania, by the creation of special State authorities with the power to borrow. In other States the credit of counties and cities is utilized in the development of urban expressways and controlled-access highways, by the issue of limited-obligation bonds secured by road-user taxes and other pledged revenues. Traditional methods of State highway bond financing are being used with conspicuous success in a number of States, among them Maryland and Massachusetts; but even in this field a choice is offered between limited-obligation and general-obligation bonds. North Carolina and West Virginia differ from other States in this group by using bond-issue funds to improve their secondary road systems.

Only time and experience can show which of these diverse methods of credit financing are most suitable in the highway field. Current examples of the success of toll-free financing of major improvements, and efforts toward the prudent and economical financing of toll facilities suggest that, in the long run, the true principles of public service will prevail over any tendency to exploit the money-making potentialities of traffic demand.

HIGHWAY inadequacies and deficiencies are almost universally recognized today—by highway engineers and administrators, by motor-vehicle manufacturers, suppliers, and users, and by the public generally. A great deal has been said in recent years about highway needs. Numerous States have made careful studies of road and street requirements within their borders; these and other data have formed the basis of a number of Nation-wide estimates of highway needs—of the Interstate Highway System (*1*),[1] of the Federal-aid system (*2*), and of all roads and streets (*3, 4*).

A few years ago there was talk of a $4-billion program, amounting in more exact terms to $4,413 million (*4*), as the annual sum required to meet all road and street needs. This estimate, based on the concept of a program to achieve adequacy in 15 years, made use of a rather optimistic forecast of the trend of highway prices, which were expected to cascade downward toward a level 50 percent above the prewar status. Actual expenditures for all roads and streets in 1950 were $4.27 billion, or nearly $4.6 billion if debt retirements are included (*5*). Thus the $4-billion program is at hand, but it has brought far less accomplishment than was contemplated in the original $4.4 billion estimate.

Of late there has been little talk of declining price levels; and a figure of $5.7 billion has emerged as the annual requirement for an adequate road and street program. That value is, in effect, the over-optimistic $4.4-billion program converted into 1951 dollars. Not claimed to be highly accurate, it will stand as a fair indicator of the extent to which our annual achievements fall short of our highway needs.

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[1] Italic numbers in parentheses refer to the list of references on page 88.

Massachusetts State Route 128 at Weston.

Difficulties in Financing

The greatest difficulties—and the widest areas of disagreement—are found in the field of highway finance. In all too many States the legislatures, or the people by popular referendum, have failed to ratify the plan for financing the engineering program of highway needs; or, by adopting half-measures, have put the improvement program on short rations. These failures, and the general Nation-wide inability to match needs with productive efforts, have caused some people to say that we are fighting a losing battle against the destructive forces and imperative demands of highway traffic. This pessimism, not to say defeatism, is unwarranted. Notable highway improvements, rural and urban, in a number of States in recent years have advanced the highway plant toward adequacy, even though not at the desired rapid rate. Two States, Michigan and Washington, have taken aggressive steps to meet the challenge of their highway needs studies. By legislative enactment (*6*, *7*) both States have launched a double-barreled attack, providing for increased highway taxes and the use of bond issues. These are only two out of numerous examples that can be cited of bold action to meet the threat of inadequate highways. And it must be acknowledged that the toll road movement, although in the opinion of many it is storing up problems for the future, is at least giving us some roads as of now.

A canvass of the States that have proceeded boldly to the attack on highway deficiencies will reveal the fact that the judicious use of public credit is a most potent weapon in this struggle. It is the purpose of this article to examine the recent history of credit financing of highways in the United States, with particular attention to those devices and procedures for borrowing that have found favor among the States, or give particular promise for future use. Toll-revenue bond financing by public authorities is a form of credit financing for highways and is treated as such herein, without discussion of the controversial aspects of the toll-road movement. The article is concerned with four major themes: (1) A discussion of the use of credit in public finance, in general, and with particular reference to highways; (2) a brief historical review of highway bond-issue financing; (3) a discussion of recent developments in the field, with statistics of bonds issued, redeemed, and outstanding during the period 1945–50; and (4) consideration of the methods of borrowing employed, the progress made, and the recent experiences encountered in a number of specific States (insofar as possible, information up to June 30, 1952, is included).

A word should be said about the current situation. The stringency in steel and other scarce metals may put a damper on the highway construction program during coming months. The inflated condition of highway prices may also act as a deterrent, although up to now the urgency of need has been the compelling factor. Finally, highway borrowing, although relatively of very modest proportions, may be charged by some with contributing to the inflationary spiral, through the expansion of credit. This article is not concerned with the immediate situation, or with short-term decisions that may have to be made about particular bond issues. It is concerned with the role of highway bond financing in an economy such as we have experienced during the past few years and seem likely to have over a considerable period in the future—one of high national income, with generally expanding production and a steady increase in motor-vehicle traffic.

PRINCIPLES OF PUBLIC CREDIT

Credit, which is the power to borrow money, permits the current use of future assets. It plays an important part in daily life. Business uses credit to finance capital improvements, to facilitate day-to-day operations, and to meet emergencies. Individuals borrow to purchase homes and for other major outlays, as well as to meet emergencies. Installment credit is widely used to finance the purchase of automobiles, household appliances, furniture, and other goods. For governments, no less than for individuals, credit is a useful tool.

To meet the varying needs of ordinary private credit operations, divers credit instruments have been developed, all of which involve the present use of or control over purchasing power in exchange for a promise to repay the amount so used. Among these are notes, bills, letters of credit, and bonds.

Public credit is a branch or form of credit in general. It enables a governmental unit to obtain something of value by promising to pay at a later time. The promise of the borrowing unit of government may bear one of several appellations depending on the term of the debt and local terminology. Among the more common are note, warrant, certificate of indebtedness, debenture, and bond. Notes, warrants, and certificates of indebtedness are usually of a shorter term than are debentures or bonds. In general investment terminology, short-term obligations are called notes and long-term obligations are called bonds. This article deals primarily with public long-term obligations incurred for highway improvements; in other words, State, county, and local highway bonds.

It has been said (*8*) that "Since public credit is simply the use by government of a device known and used throughout the commercial world, the principles underlying government credit are not different from those that apply generally." Both public and private credit depend upon the resources and reputation of the debtor. The principal differences between public and private credit stem from the security behind the loan. In the case of public revenue bonds not backed by the full faith and credit of the issuing government, even this difference diminishes.

Prudent Public Borrowing

The circumstances under which it is prudent for a government to incur debt do not differ greatly from those under which it is proper for an individual or a corporation to borrow money. The principle that it is advisable to borrow when the use of money is more valuable at the time of borrowing than at the time of repayment holds true for goverments as well as individuals. Howard (*9*) states the matter in these words: "If the earlier use of money is more valuable than the interest that must be paid, then, as a general rule, it is unobjectionable for the government to borrow. Indeed, it may not only be wise and prudent but also highly necessary. So far as the use of money is more valuable at a given time than its interest cost, there is, then, no pronounced difference between public and private credit."

It is well to examine this principle for a moment, as it lies at the root, not only of the issuance of bonds, but of all monetary transactions. The matter at issue is the time value of money. Without entering into the basic economics, or the psychology, of interest and interest rates, we may state that money in hand is always of more value than an equal amount in future prospect. A discussion of this principle as it affects the entrepreneur is found in Baumol's *Economic Dynamics* (*10*).

If P_1 is defined as a certain amount of money expected at the end of a period, say a year, and P_0 is its present value (a smaller amount), we may define the interest rate i as such that:

$$P_0(1+i)=P_1.$$

If P_2 is the amount in prospect at the end of the second year, having value P_1 at the end of the first year, we have:

$$P_1(1+i)=P_2;\ \text{thus}$$
$$P_0(1+i)^2=P_2.$$

By inspection it follows that if P_n is defined as an amount of money expected at the end of n years, its present value is given by the equation:

$$P_0(1+i)^n=P_n;\ \text{and}$$
$$P_0=\frac{1}{(1+i)^n}P_n.$$

If the quantity $1/(1+i)$ is defined as the discount factor D we have:

$$P_0=D^nP_n.$$

The above is no more than a statement of the principle of compound interest; but it should be borne in mind, because there are those who maintain that interest on a government debt is merely so much money wasted. The crux of the matter lies in the advantage gained from money in hand rather than in prospect.

Value of Money in Hand

In a simple example of private enterprise, a man may wish to borrow money to build a factory, in anticipation of profits to be had from the manufacture and sale of a commodity. Since he cannot build the factory without it, money in hand is of great value to the would-be manufacturer. The capitalist, knowing the value of his money and wishing to derive an income from it, appraises the risks involved as well as the relative time values of money; and reaches an agreement with the borrower as to the rate of interest.

To a government wishing to borrow money the advantages of money in hand rather than in prospect are seldom in the form of anticipated profits, although they often have to do with the operation of public enterprises. The advantages exist, nonetheless, and they are the only reason why a governmental unit should float a loan and pay interest on it. Sometimes the advantage takes the form of relief from a dire emergency or from temporary financial embarrassment; at other times great and lasting benefit to the community is anticipated. To the latter category belong the highway bond issues with which this article is concerned.

For governments as well as individuals and corporations the proper function of credit is to serve as a supplement to current revenue. The principal of any debt, together with interest, must ultimately be paid from current revenue unless the debt is defaulted or repudiated. Although failures to repay public debt have not been unknown in American history, highway borrowings in the modern era (1890 to date) have been conspicuously free from this blight. The necessity for the repayment of loans out of taxes does, however, bring to the fore the fact that there are three parties to a public credit transaction: The governmental unit, acting in the role of management; the taxpaying public, which either directly or through legislative action, must approve the loan; and the money lenders, or investing groups.

Time Value of Money

The attitude of each of these three parties is affected by views regarding the time value of money—the value of money in hand in relation to money in prospect. The public in particular, tax conscious as it generally is, must decide whether to forego the prospective advantage altogether, to finance it out of current—and perhaps very burdensome—taxation, or to finance it by means of a loan, e. g., to make the payment out of dollars of which the present value is less than that of the dollars out of which current taxes must be paid. The governmental body, representing the public, attempts to act in its interest. The investing groups, in appraising the desirability of the proposed loan, must weigh not only the relative time values of money but also the lower risks involved in municipal as compared with industrial securities, and certain income-tax advantages to be derived from the possession of tax-exempt bonds.

Since public borrowing is, in effect, postponed taxation, the answer to the problem of when it is financially proper for a government to borrow requires consideration of the necessity or equity of distributing part of the financial burden to future taxpayers. Shultz, in his work *American Public Finance* (*11*), has stated: "Authorities on public finance have established four fiscal purposes which may justify governments in raising funds by borrowing—to finance large emergency or irregular expenditures, to finance capital construction projects, to harmonize the divergent rhythms of current expenditures and current revenues, and to refinance existing debt. Certain nonfiscal considerations may occasionally provide supplementary justification for a government's borrowing policy." Before proceeding to an examination of the special features and attributes of borrowing for highways, it seems expedient to discuss in general terms these four accepted justifications of credit financing.

Emergency Expenditures

In the case of unforeseen or unavoidable emergencies the resort to borrowing may not only be wise and prudent; it may be inevitable. The practice of the Federal Government in financing a large part of its war expenditures by the issue of bonds is a prime example of emergency borrowing. On the State and local level, floods, earthquakes, and other disasters not infrequently occasion large expenditures which can most conveniently be covered by means of loans. State and local governments that have neglected, or have been unable to keep institutional and other facilities abreast of the needs of modern times, may find the urgent need for rehabilitation constituting an emergency justifying the resort to credit financing. The need of our highways for modernization to meet traffic demands is a special instance of this; but this category, relating as it does to public works, belongs more properly in the second of the four justifications for borrowing.

The New Hampshire Turnpike: interchange and tollhouse.

Capital Outlay

Writers on public finance (*8*, [illegible], *11*) are generally agreed that capital outlays for self-supporting public enterprises are properly financed by borrowing. Such enterprises include city water departments, power authorities, irrigation developments, and, of course, highway toll facilities. (There is much current controversy about power authorities and other water-resources developments, as well as about toll facilities; but it should be observed that the arguments center about the desirability of the enterprises themselves, and not about the use of credit to finance a self-supporting enterprise.) As in the case of a private corporation issuing bonds to create or acquire property, the bonds are often technically mortgages against the property of the enterprise; but the real security behind the issue is the assurance that revenues will be more than sufficient to maintain the property and to pay interest on the bonds.

Most authorities are willing to carry the analogy a step further by sanctioning the use of public credit to finance long-lived improvements such as schools, hospitals, and other institutions, and roads and bridges—providing, of course, that the improvements will outlast the term of the bonds. On this subject Lutz (*8*) has the following to say: "If the debt maturities are arranged in accordance with the probable life of the improvements, and if the bonds are always redeemed and never refunded, a case can be made out for permitting the necessary funds to be raised by means of loans. The advocates of the loan policy for such improvements will contend that the taxes required to pay interest and sinking-fund charges on the unredeemed installments of the debt would be balanced by the loss to taxpayers that would result if a very much larger tax levy were made outright in order to pay for the improvements in cash as they are constructed. If the outlay is so large that the tax would seriously retard industry and initiative this position is doubtless well taken. Credit enters as the useful and necessary supplement to current revenues for the purpose of spreading the burden over time, and so, in reality, of lightening it."

Although cautioning against the indiscriminate use of borrowing to finance all capital outlays, students of public finance are almost unanimous in agreeing that extraordinary capital outlays for improvements that will benefit future taxpayers are properly financed by borrowing. Adherence to this concept requires that a distinction be made between ordinary continuing capital expenditures and extraordinary capital expenditures. Practically all governmental units make some capital outlays each year to provide for normal growth and to replace existing capital improvements. To the extent that such capital expenditures are an ordinary and continuing phase of governmental activities, as are expenditures for maintenance and operation, they should be provided for from current revenues. On the other hand, large capital outlays of a type which will not soon be repeated cannot always be financed from the revenues of a single year without throwing an inordinate burden on the taxpayers. If, however, such outlays are financed by borrowing, the taxes necessary to service the debt can be spread over a period of years and the increment of the construction cost in each year's levy will be too small to be irksome. Furthermore, future taxpayers who contribute towards the retirement of the construction debt will be deriving a benefit from it. Under such circumstances, borrowing for the construction of public works which are socially advantageous and productive is a proper use of public credit.

The Delaware Memorial Bridge.

The Cyclical Budget

The issuance of short-term notes or certificates of indebtedness to balance the irregular flow of revenues in relation to disbursements is a common, although not universal, practice at all levels of government. Of greater significance is the concept of a cyclical budget which has developed in part from the depression experience. This concept (*12*) emphasizes "the desirability of increasing public spending and reducing taxes in hard times as measures to soften depressions and to promote economic recovery. It follows that borrowing will be called upon to close the gap between expenditures and tax receipts. * * * When economic conditions become favorable, compensatory fiscal policy will require increases in taxation and a reduction of indebtedness."

Highways participated in the counter-cyclical effort during the depression years. A number of States (among them Massachusetts, New York, and Washington) and many local units issued public-works bonds for unemployment relief, the proceeds being used in part for highway work. The Federal Government utilized its borrowing power to establish a loan and grant procedure under the Public Works Administration and for a work-relief program under the Work Projects Administration and its predecessors. Each of these agencies in its own way spent a considerable amount of money in highway work. Grants to the States through the agency of the Bureau of Public Roads were also increased, although on a nonmatching basis. These efforts, although very useful in providing unemployment relief, did not result in a major counter-cyclical effort, as they served largely to replace dwindling State and local revenues.

World War II interrupted the supposedly normal cycle of depression and prosperity, causing an abrupt curtailment of highway building and other peacetime public-works expenditures in the face of tremendously increased deficit financing. During the postwar period it has been impossible to catch up with the need for highways, schools, and other public improvements because of the demands of increased population, increased industrial activity, and increased spending power. Thus the conditions favorable to the application of a cyclical-budget policy have not come into being.

Refinancing

Borrowing for refinancing embraces both refunding and conversion. These terms are not identical, although they are often used interchangeably without distinction. Generally speaking, refunding is the postponement of debt payment, while conversion is a reissuance to provide for different (usually lower) interest rates or details other than the postponement of debt maturities. Both refunding and conversion may be combined in a single operation, called by either name.

The necessity to refund may indicate poor management of the debt or the occurrence of an unforeseen emergency that threatens to interfere with amortization of the debt. A general easing of restrictions on refunding was necessary during the depression to lessen the large volume of defaults. Conversion, on the other hand, is usually employed to reduce interest rates, with a resultant saving in the costs of debt service. The burden of public debt was not greatly reduced by conversion until the occurrence of unusually low interest rates which followed the depression of the early 1930's and still persists. This situation has resulted in the exchange of new securities at lower interest rates for outstanding obligations by many corporations and governmental units.

PRODUCTIVE BORROWING FOR HIGHWAYS

The photographs illustrating this article demonstrate current activity in highway building financed out of bond issues. The pictures and their captions speak for themselves and for the States that have undertaken the work, serving as indicators of progress in the struggle for adequate highways. Many of the photographs appear through the courtesy of the State highway departments.

Of the four justifiable purposes of public borrowing discussed in the preceding section, we are concerned in the highway field chiefly with the second—borrowing for capital improvements; and largely as a corollary of this, with the first—borrowing for emergency or irregular expenditures. The road and street plant is in such a condition today with respect to the demands of traffic upon it that in many States a program of extraordinary expenditures is needed. The financing through bond issues of such an emergency program of capital outlays is thus doubly justified.

The Tacoma Narrows Bridge in Washington.

User Taxes in Relation to Credit Financing

Because of the practice of dedicating road-user tax revenues for highway purposes, highways occupy an intermediate position between the self-supporting public enterprises and entirely tax-supported activities, such as the schools. This is particularly true of State highways, which in all States are supported almost entirely by State highway-user imposts supplemented with Federal-aid funds. Even though most State highway bonds are secured by the full faith and credit of the State, the interest and principal payments on State highway bonds are almost universally made from the road-user tax receipts. It is therefore appropriate to explore the implications of viewing highways, or at least the primary highway systems, as governmental enterprises supported by charges upon the users. Upon this basis the proposal for improvement by credit financing of a system or group of roads would be justified if the revenues generated or "earned" by the traffic on the particular group of roads will be sufficient, or more than sufficient, to defray the debt service charges and the costs of maintenance and operation.

The idea that bonds issued to finance arterial highway improvements should be secured by a pledge of their annual earnings in road-user taxes has been advanced by a number of authorities (*13, 14, 15*). In calculating such earnings the ordinary procedure is to evaluate the traffic on a given road or group of roads in terms of the annual amount of motor-fuel tax revenues generated, by the use of reasonable values of miles per gallon for the different types and sizes of vehicles of which the traffic is composed. To this may be added a pro rata of annual registration fees and miscellaneous motor-vehicle receipts. Other available revenues, including Federal aid, should be accounted for in the calculations.

Not everyone would agree that the earnings of specific highways or groups of highways should be pledged to the service of bonds issued for their improvement, although the more generalized dedication of road-user tax revenues for debt service on State highway issues is the common practice. Perhaps a more pertinent procedure, with respect to any proposed highway bond issue, is to determine at what rate of user taxation (together with other available revenues) the facilities to be improved will generate sufficient revenues to pay off the issue and provide for maintenance and operation.

Highway Toll Facilities

Toll roads and bridges are, or purport to be, supported by their toll revenues plus any additional income, such as receipts from concessions, that may be provided for. This being the case, they fall into the category of self-supporting public enterprises. It is not intended here to enter into the controversy regarding the modern toll-road movement. It may be noted, however, particularly in the case of toll roads, that a higher rate of interest or, to be more exact, a higher yield is generally exacted by the investors than in the case of ordinary highway bond issues. With some exceptions, the faith and credit of the State is not pledged, and State user-tax revenues are not available for debt service. With the toll revenues in effect the only security, an element of greater risk is recognized in the higher yield.

Conditions Favorable to Bond Financing

It is an accepted principle of public finance that capital outlays for replacements and to ment the needs of normal expansion should be met from current revenues rather than from borrowing. Capital outlays for highways are needed each year to replace or reconstruct worn out highways and to provide for normal increases in traffic. Such continuing capital outlays are properly a current expense, like maintenance and administration, and should be provided for from current revenues. Prolonged contraction or interruption of this process of replacement and improvement, however, will inevitably result in the accumulation of a backlog of needed construction. If the additional expenditures required to remedy the accumulated deficiencies are too great to be provided from current revenues within a reasonable time, the situation is an emergency warranting resort to borrowing. Even if the cost can be met by a long-term current-revenue program, shortening the time required for rehabilitation and modernization by credit financing is a proper and justifiable use of public credit, provided the benefits from earlier improvement are at least equal to the interest costs incurred.

It is needless to labor the point that we are in just such a situation today. With an accumulated backlog of needed improvements inherited from the war years and increasing each year with the mounting demands of traffic, the need for an accelerated highway program is recognized in almost every State. The rate of highway expenditure can be accelerated by an increase in highway taxes; and this is what should be done if the intention is simply to move to a higher level of normal expenditures. A truly accelerated program, however, contemplates a short period of abnormally high capital-outlay expenditures, during which the highway plant will progress rapidly toward a condition of adequacy. Such a program can best be financed by a bond issue.

The advantages to be derived from such an accelerated program lie in the reduction of vehicular operating costs, including a reasonable assignment of values to time costs, of accidents and their costs, and of the strains and discomforts of driving over inadequate highways. Since all of these benefits will be realized at earlier dates under an accelerated program, their accumulated values over a given time will be much greater than under a long-term current revenue program.

The design of a highway improvement program should be selective. Credit financing is best adapted to facilities having a relatively long investment life. For that reason, and also because of the urgency of need, it is natural to plan first for the use of bond issues in the construction of arterial improvements, both rural and urban. Close study, however, may reveal that the improvement of other systems may advantageously be financed out of bond issues.

Credit Versus Pay as You Go

The choice between credit financing and "pay-as-you-go" (which may well be called "pay-before-you-go") is essentially a choice as to the rapidity with which the desired capital improvements will be made. The increased costs, if any, of credit financing must be weighed against the benefits that will be derived from earlier completion of the project or program. As Howard (*9*) puts it: "One misstatement, repeated again and again, is that in return for the principal only, the Government pays back the principal plus interest. This is a reprehensibly incomplete statement of the facts. The Government obtains the principal at the beginning of the transaction, plus the use of other people's money throughout the term of the bonds, in return for which it pays the interest during the period of the loan plus the principal at the end of the term. The use of other people's money is as valuable as, or presumably more valuable than, the interest paid for that use; otherwise the money would not be borrowed."

Credit financing does not, however, inevitably increase the cost to the governmental unit (and therefore to the taxpayers) by the amount of the interest costs incurred. Under a "pay-as-you-go" program of highway improvement extending over a long period of time, some sections of road will require temporary or stop-gap improvements while awaiting final improvement to the desired standards. An accelerated program of improvement made possible by borrowing may, under certain conditions, reduce the amount of stop-gap and temporary improvements enough to offset part or all of the interest cost.

Bertram H. Lindman, who acted as consultant to the Ohio highway fiscal and tax study in 1950–51, suggested the possibility that the 20-year highway improvement program recommended for all roads and streets in that State could be modified by introducing a 10-year bond-issue program for the State A and B systems (rural and urban) and retiring the issue during the second 10 years, without increasing the annual revenue requirements of the total road and street program. At Mr. Lindman's request, calculations were made at the Bureau of Public Roads to test the idea; and these calculations, subject to the soundness of the given data, confirmed the validity of his suggestion. His findings and the supporting calculations were subsequently published by the Ohio Department of Highways (*16*).

A key figure in the analysis was supplied by the engineering consultants for the Ohio highway needs study, who estimated that the annual requirements (construction, maintenance, and administration) of the State A and B systems, which would amount to nearly $184 million during the 10-year bond-issue program, would drop to less than $62 million during the second 10 years. Table 1, taken from the Lindman bulletin, gives the essential steps in the calculations made in connection with this unusual proposal. It will be observed that a bond issue of about $607 million is required; and that it is retired, with interest at 2 percent, during the second 10-year period, with a $7-million average annual surplus of revenues over requirements.

The Ohio calculations illustrate one of the possible dangers of too-great reliance on credit financing. The greatly accelerated program would produce a profound lull in construction activities in the second 10 years, during which replacement needs would very slowly accumulate. Such a great discontinuity in construction activity might be objected to on the ground that it would disrupt both the staff of the State Department of Highways and the contractors' organizations.

Table 1.—Bond-issue proposal for the State of Ohio [1]

Program period:	
January 1, 1953, to December 31, 1972; bonds issued in 10 equal annual installments, starting July 1, 1953, and retired in 10 equal annual payments starting July 1, 1963.	
Requirements:	
State system 10-year program cost	$1,838,560,000
Accomplishments to Dec. 31, 1952	100,000,000
Net total program cost	1,738,560,000
Net annual requirements, State system	173,856,000
Evaluation of required bond issue for State systems:	
B=amount of required bond issue.	
E=required annual expenditure, exclusive of interest on bond issue=$173,856,000.	
i=annual interest rate, assumed at 2 percent.	
I=average annual interest payment, 1953–62=$\frac{B}{2}i=0.01B$.	
R=average annual revenues available=$119,249,000.	
$R+\frac{B}{10}$=average annual income, 1953–62.	
$E+I$=required average annual expenditures, 1953–62.	
$R+\frac{B}{10}=E+I$.	
Substituting, $\$119{,}249{,}000+\frac{B}{10}=\$173{,}856{,}000+0.01B$.	
Thus, B=$606,744,000=required bond issue.	
Transactions of second 10 years, 1963–72:	
Average annual expenditures, exclusive of debt service, 1963–72	61,908,000
Annual retirements	60,674,000
Average annual interest requirements	6,067,000
Average total requirements	128,649,000
Average annual revenues for State systems, 1963–72	135,745,000
Average annual surplus	7,096,000

[1] Source: Bertram H. Lindman, *Supplemental Bond Financing for Acceleration of the Ohio Highway Program*, March 1951 (prepared for the Ohio Program Commission), p. 28.

A program actually prepared for adoption would probably be less drastic in this respect. This factor of discontinuity is, however, a practical consideration which in the absence of other restraining factors would tend to set a limit on the extent of an accelerated bond-issue program.

HISTORICAL BACKGROUND [2]

In America the earliest public borrowing was done by the English colonial governments. These loans were primarily to provide funds for current expenses and to supply a circulating medium. Borrowings to obtain funds for capital outlays were of minor importance. During and after the Revolution the State governments, successors to the colonial governments, borrowed for both war and civil current expenses. After ratification of the Constitution and the establishment of the Federal Government the States discontinued further borrowing and, although a few States negotiated bank loans during the War of 1812, large-scale borrowing was not begun until the 1820's.

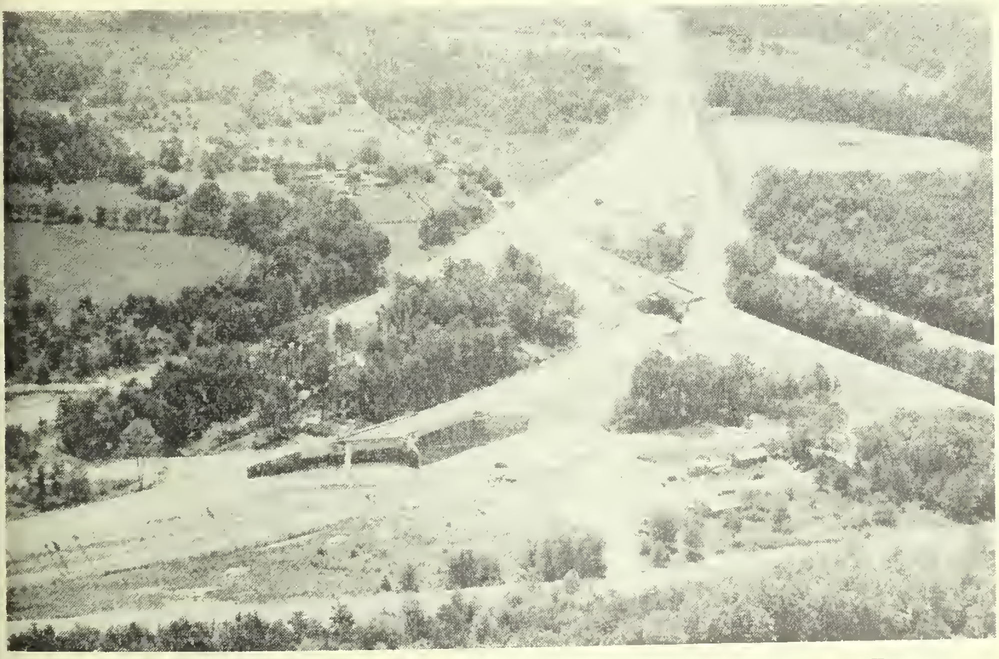
Early Borrowing for Internal Improvements

The early State debts were almost entirely in the form of paper money and certificates. Many were, in effect, forced loans. With the advent of borrowing for internal improvements in the 1820's, State bonds made their appearance in the investment market, marking the beginning of State debts as they are known today. State borrowings for permanent improvements began in this country some time after 1820; and by 1838 the States had issued $175 million of bonds to construct or subsidize canals, turnpikes, railroads, banking, and other ventures, of which about $10 million were for turnpikes.

The panic of 1837 brought an abrupt end to the States' borrowing for such internal improvements and in the early 1840's nine States (Arkansas, Florida, Illinois, Indiana, Louisiana, Maryland, Michigan, Mississippi, and Pennsylvania) defaulted and four others (Alabama, New York, Ohio, and Tennessee) barely avoided it. Previous to 1840 no State constitution limited the debt which the legislature might incur, but the crisis years of the 1840's brought about a natural and expected reaction and, by 1857, 19 States had adopted constitutional limitations on borrowing. These amendments were part of a general movement by the voters to withdraw some of the powers liberally granted to legislatures by early constitutions. Debt limits were usually included in the constitutions of States admitted to the Union after the Civil War.

Renewed demands for State aid to internal improvements accompanied the improvement in business conditions and the recovery of State credit, and a second, but smaller, borrowing boom was under way by 1850. During this period none of the States which defaulted in the 1840's participated; only Southern and Western States (and New York) borrowed; borrowing was largely for railroads; and the borrowing was slower and more cautious than in the 1820's and 1830's. Except to issue bonds for war purposes during the Civil War, the States borrowed little during the remainder of the nineteenth century. By 1900 only a little over $12 million of State highway bonds had been issued. Beginning in 1890 Idaho had issued about $200,000, and Massachusetts, beginning in 1893, had issued the remainder. By 1912 the States had issued $67 million of highway bonds.

Historical data concerned with the borrowing habits of the counties and other local rural units for highway purposes are not readily available. It has been estimated that in 1870 the counties had a total net debt outstanding of $188 million. It is believed that very little of this debt was for highway purposes. As nearly as can be estimated these units had highway bonds of approximately $10 million outstanding at the turn of the century. They increased their rate of borrowing rapidly, and by the end of 1912 had issued approximately $200 million in highway bonds.



Construction of the airport interchange in Maryland: Baltimore-Washington Expressway.

The urban units of the United States, as nearly as can be ascertained, entered the credit financing field around 1830. By 1840 it has been estimated that the total municipal indebtedness for all purposes was $20 million. Municipal debt mounted rapidly through the next two decades and by 1860 was estimated at $200 million.

The decade following the Civil War was an era of great industrial expansion and the urban units borrowed heavily to finance various enterprises in an effort to keep pace with private enterprise. They subscribed freely for railroad stock and borrowed on a large scale for paving, sewer, and water-supply improvement. By 1880 the urban units were in debt to the tune of $725 million—three times that of the States. How much of this indebtedness was incurred for street purposes is unknown, but it was undoubtedly a fairly substantial amount.

Borrowing for Highways

The advent of the motor vehicle brought a demand in the early part of the twentieth century for improved roads on which to use this form of personal transportation. Improved highways, in turn, increased the demand for private motor vehicles and permitted development of the motor-vehicle transportation industry. Thus, the motor vehicle and the highway on which to operate it are complementary and development of each increases the demand for the other.

Borrowing for State highways since 1921 has been characterized by two periods of relatively large-scale activity. The first coincided with the large road-building program of the 1920's when many States borrowed heavily for original State highway construction or assumed large debts of counties and other local governmental units in return for roads incorporated into the State system. The depression and war periods saw a lessening of borrowing, although during the early 1930's further State assumption of local debt, and revenue bonds used to finance the construction of large toll facilities, raised the total outstanding debt for State highways in 1938 to what remained an all-time height until 1951. Borrowing since 1946 has resumed the same upward trend exhibited during the 1920's.

Relation to State Construction

The relation of State borrowing to the total State highway construction program since 1920 is shown in table 2 and figure 1, which

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[2] Much of the historical background cited here was obtained from references *11, 17, 18*, and *19*.

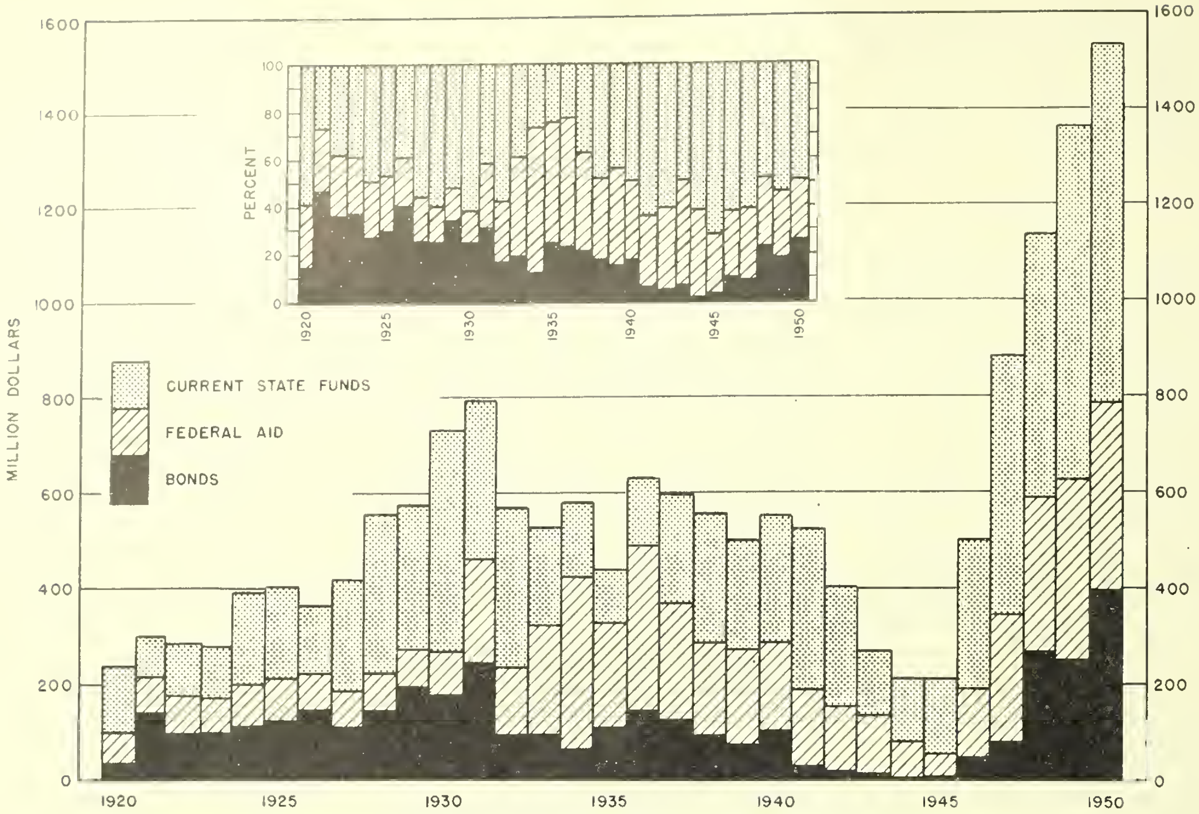


Figure 1.—Cost of State highway construction analyzed by source of funds, 1920–50.

depict three sources of revenue principally used for State construction: proceeds of bonds, Federal-aid, and current State funds.

Table 2.—Source of construction funds for State highways, 1920–50 [1]

Year	Construction bonds issued [2]	Federal funds received	Current funds used	Total construction expenditures
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
1920	36, 607	61, 966	141, 767	240, 340
1921	139, 915	77, 741	82, 953	300, 609
1922	96, 802	79, 741	110, 918	287, 461
1923	98, 167	73, 308	108, 518	279, 993
1924	107, 398	92, 970	197, 280	397, 648
1925	120, 672	92, 343	190, 828	403, 843
1926	142, 684	79, 163	144, 164	366, 011
1927	106, 229	80, 160	232, 431	418, 820
1928	142, 338	81, 252	334, 891	558, 481
1929	197, 434	77, 952	300, 089	575, 475
1930	179, 252	94, 111	455, 524	728, 887
1931	242, 974	218, 383	335, 545	796, 902
1932	98, 197	138, 857	332, 457	569, 511
1933	97, 919	223, 586	205, 507	527, 012
1934	66, 583	351, 812	158, 974	580, 369
1935	109, 828	219, 381	109, 097	438, 306
1936	145, 263	346, 281	140, 216	631, 760
1937	124, 654	245, 749	218, 839	589, 242
1938	97, 767	189, 908	270, 704	558, 379
1939	75, 552	202, 714	221, 847	500, 113
1940	102, 119	185, 192	269, 820	557, 131
1941	30, 641	160, 526	334, 066	525, 233
1942	20, 475	136, 266	244, 953	401, 694
1943	18, 160	119, 254	131, 281	268, 695
1944	3, 000	77, 379	129, 949	210, 328
1945	6, 500	53, 145	150, 822	210, 467
1946	51, 378	139, 025	311, 913	502, 316
1947	79, 917	262, 474	539, 960	882, 351
1948	265, 932	324, 802	547, 940	1, 138, 674
1949	248, 937	377, 943	735, 070	1, 361, 950
1950	397, 569	385, 191	751, 099	1, 533, 859

[1] Adapted from Public Roads tables SF-201, SF-202, SF-3, and SB-202.

[2] Includes bonds of the local units which were assumed by the States in the same year the bonds were issued.

This is not the whole story, however, as the current funds in the earlier years include contributions from the counties and other local units, largely from their own bond issues, for State highway construction.

Throughout the period from 1921 to 1930 proceeds of bonds contributed from 25 to 47 percent of all State construction funds. For the 10-year period bond proceeds amounted to 31 percent of aggregate funds expended for construction. In several States more than half of all construction funds represented bond issues. For example, in North Carolina the entire State highway development program was based on a bond authorization of $115 million. This program resulted in the modernization of the State's highway system in the decade ending in 1930 when 3,500 miles of road were paved for the first time. The bonds issued were backed by the full faith and credit of the State and accounted for 70 percent of all money expended on highway construction during the decade.

Similarly, Illinois and Missouri utilized productive borrowing to accelerate their highway-development programs. In Illinois two bond authorizations totaling $160 million contributed 60 percent of all construction funds from 1921 to 1930. Missouri borrowed $135 million in a similar period and greatly accelerated its highway development. Numerous other States issued bonds during this period, and still others utilized the credit of the counties and other local units in the construction of what became their State highway systems.

The decade from 1931 to 1940 produced borrowing at a slightly lesser rate, even though there were large assumptions of local debt and huge revenue issues for such facilities as the San Francisco bridges and the Port of New York Authority's bridge and tunnel program during this period. Federal aid assumed a dominant role in financing during the depression years with increased authorization for work programs. During the period 1931–40, Federal funds accounted for 40 percent of all construction moneys while bond proceeds contributed one-fifth of all construction funds.

From 1941 to 1950, there were two contrasting developments—the cessation of normal highway construction during World Wa II; and the postwar construction boom, characterized by a rapid increase in highway borrowing.

The fact that borrowings were used extensively in meeting the urgent and vociferou demand for improved roads that permitte and accompanied the development of motor vehicle transportation is not surprising. Public revenues from traditional sources, principally the property tax, were not capable o expanding rapidly to meet the demands of thi new form of transportation, and the newl adopted highway-user taxes could develop a an important source of revenue only as improved highways permitted realization of th increasing desire for motor-vehicle ownershi and use. There is no doubt that the judiciou use of credit financing during the early perio of modern highway transportation not onl resulted in savings to vehicle operator through reduced operating costs, but also accelerated the growth of highway transportation. Quoting Edna Trull (*20*): "There wa dequate justification, moreover, for this type f financing. The immediate benefits of fa-ilitating communications and providing conomies in transportation were obvious, and he improvements were expected to have long-erm value."

Total Borrowing for Highways

Figure 2 shows the sale of bonds for high-ay purposes by the States and by the coun-ies and other local rural units in each 5-year eriod from 1901 to 1950. Table 3 gives the mounts of State highway bonds and of local ural highway bonds issued in each year from 901 through 1950.

Similar information is not available for he incorporated places: Street finance data ere not compiled in segregated form until he mid-1930's when the highway planning urveys in the various States started collect-ng such information. The net street debt of he urban units, as reported in the fiscal studies prepared by the highway planning surveys, amounted to $1.5 billion in 1935. By 1945 the gross street debt outstanding was estimated to be $1.2 billion. Since that time it has increased until at the end of 1950 it is estimated to have again reached $1.5 billion.

able 3.—Highway bonds issued by the States and local rural units, 1901–50 [1]

Year	State highway bonds			Highway bonds issued by counties, towns, townships, etc.
	Original issues	Reimbursement obligations assumed	Total	
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
1901	350	-------	350	2,245
1902	900	-------	900	1,947
1903	605	-------	605	3,441
1904	450	-------	450	4,638
1905	450	-------	450	3,914
1906	1,036	-------	1,036	2,996
1907	2,042	-------	2,042	6,650
1908	6,095	-------	6,095	9,124
1909	7,185	-------	7,185	21,970
1910	6,846	-------	6,846	11,367
1911	14,214	-------	14,214	18,541
1912	13,366	-------	13,366	22,672
1913	28,759	-------	28,759	35,965
1914	11,684	-------	11,684	44,285
1915	25,319	-------	25,319	53,513
1916	4,809	-------	4,809	79,731
1917	20,835	38	20,873	58,170
1918	7,029	54	7,083	35,299
1919	32,939	683	33,622	227,892
1920	33,778	3,894	37,672	128,861
1921	130,393	20,234	150,627	223,446
1922	86,050	18,164	104,214	182,626
1923	90,636	14,986	105,622	143,543
1924	97,935	27,062	124,997	157,089
1925	117,905	5,174	123,079	167,211
1926	138,121	31,706	169,827	184,723
1927	92,886	92,164	185,050	191,851
1928	118,352	25,476	143,828	169,361
1929	175,048	114,170	289,218	123,429
1930	147,964	32,301	180,265	105,318
1931	227,477	123,729	351,206	92,511
1932	95,015	107,891	202,906	62,717
1933	96,279	26,539	122,818	17,582
1934	66,283	3,592	69,875	38,179
1935	99,269	12,129	111,398	38,679
1936	136,717	10,013	146,730	49,486
1937	110,771	18,625	129,396	46,722
1938	93,101	13,989	107,090	49,010
1939	73,314	3,560	76,874	34,548
1940	100,884	8,190	109,074	20,090
1941	29,709	4,456	34,165	31,401
1942	20,475	3,970	24,445	18,861
1943	18,160	5,419	23,579	5,536
1944	3,000	37	3,037	5,892
1945	6,500	4,397	10,897	22,184
1946	51,378	3,150	54,528	49,207
1947	78,550	1,486	80,036	106,197
1948	265,614	2,458	268,072	85,826
1949	248,537	5,525	254,062	98,559
1950	396,101	4,023	400,127	104,342

[1] Refunding bonds not included.

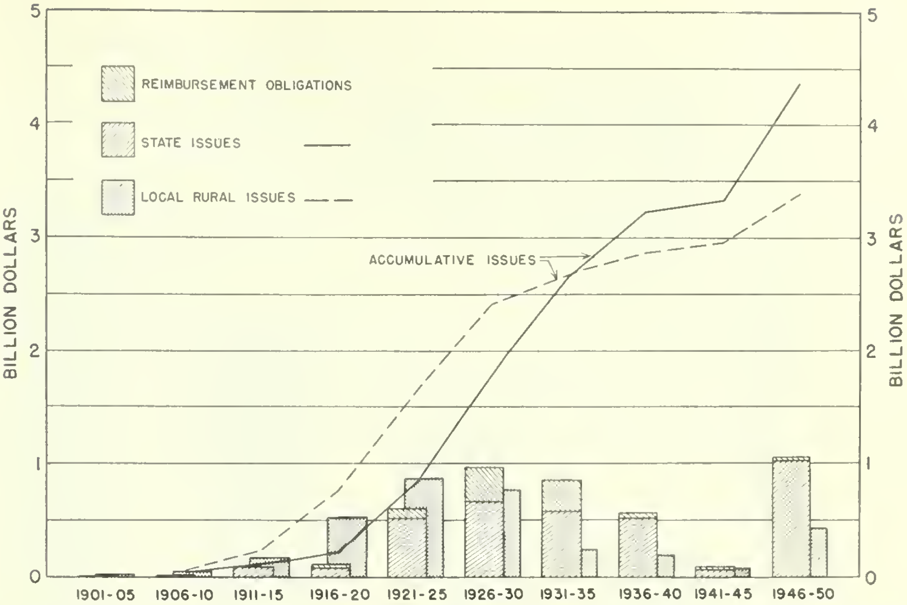


Figure 2.—Highway bonds issued by the States and local rural units, 1901–50.

The accumulated borrowings from 1901 to the end of each 5-year period are also indicated in figure 2. The outstanding debt, which is not shown on the chart, is much less than the accumulated borrowings, of course. At the end of 1950, the outstanding highway debt of the States was $2,141 million and that of the counties and other local rural units was $904 million. Despite heavy borrowings during 1946–50, the outstanding State debt is less than half the total amount issued since 1900, and outstanding debt of the counties and other local rural units is but little more than one-fourth of the total amount issued since 1900.

It is not surprising that the ratio between the debt outstanding at the end of 1950 and the total debt issued since 1900 is smaller in the case of the counties and other local rural units than in the case of the States. The bulk of the county debt was incurred earlier than was the bulk of the State debt and, therefore, much more of it has matured and been retired. It is of interest to note that the two cumulative curves cross in the mid-1930's. During the early years county and local issues dominated the field, and only gradually gave ground to the mounting total of State highway bond issues.

Reimbursement Obligations

During the early years of highway development, the counties and other local rural units borrowed both for local roads under their jurisdiction and for intercommunity roads which were then, or later became, State highways. As State highway departments were created and the States assumed more financial responsibility for the more important through highways, State borrowings increased. Also, some States undertook to reimburse the local units for their contributions to the cost of State highways. These pledges or reimbursement obligations are segregated from State bond issues in figure 2.

In general, reimbursement obligations are of two kinds: pledges by the State to pay principal and/or interest charges on local road obligations issued for State highways; and pledges by the State to refund to the local road authorities certain specified amounts over a period of years. The circumstances surrounding the assumption of reimbursement obligations vary from State to State. In some States—among them Iowa, Minnesota, South Carolina, and Wisconsin—the counties were encouraged to issue bonds and turn the proceeds over to the State with the understanding that the State would provide funds for the payment of principal or principal and interest on the bonds. The State thus used the credit of the local governments to borrow for State highways. This practice was followed in Iowa as late as 1941 and is still followed in Wisconsin. Neither of these States has any direct State debt for highways.

In other States—as in Arkansas, Kansas, and Texas—no commitment was made, at the time of the local contribution, to reimburse the local units, but later the State assumed this obligation. In some other States, among them Tennessee, the circumstances leading to the assumption of reimbursement obligations partook of both of the situations described above. In almost all cases, reimbursement obligations stem from inability or reluctance on the part of the State to assume complete financial responsibility for main highways during the early days of highway development, or to incur debt. In some cases the actual assumption of reimbursement debt was spurred on by the economic plight of local governments during the early 1930's. Although during the last two decades the States

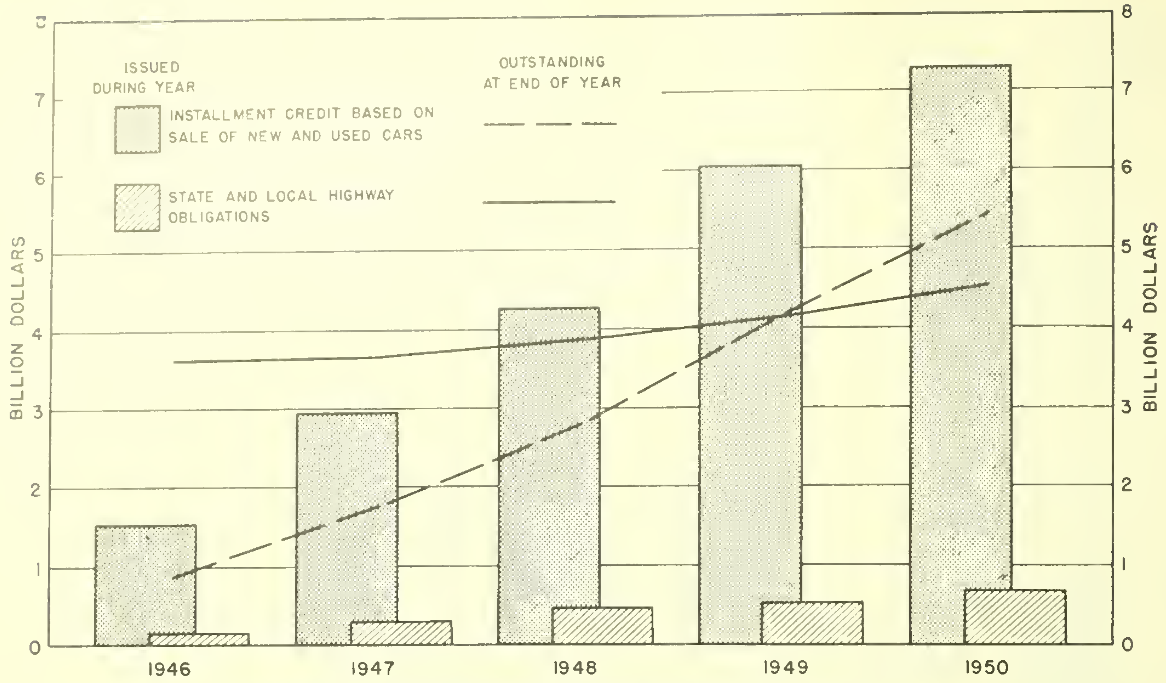


Figure 3.—Borrowings for passenger cars and for highways.

have assumed the dominant role in borrowing for rural roads, both State and local borrowings played an important part in the early years of highway development by helping to meet the emergency created by the demands of the rapidly growing motor-vehicle transportation industry for better roads.

Need for Modernization

The years since the end of World War II have brought many problems similar to those encountered during the early years of highway development.

The drastic curtailment of construction and the reduction, because of increasing price levels, in the effectiveness of maintenance expenditures during the years of World War II did much to hasten the deterioration and obsolescence of our highway systems. The demand for motor-vehicle transportation, frustrated during the war years, appears to be far from satiation. Motor-vehicle registrations and use are now increasing rapidly. Expensive highway improvements are urgently needed to serve the increasing traffic. Unfortunately, the vehement demands of highway users for highway improvement and modernization are not always matched by their willingness to pay for the improvements through increased taxes.

The magnitude of the problem facing the highway administrator is indicated by the estimate that the 1950 rate of construction was roughly 60 percent of the rate required to meet total highway needs over a 15-year period (*21*).

Much has been accomplished in spite of the skyrocketing costs of construction and maintenance. Maintenance, administration, general overhead costs (including that of highway police), and interest have taken almost 50 percent of the $16¼ billion of current revenues available for highway purposes in the period 1946–50, with maintenance alone requiring 37 percent.

It is true that revenues have increased tremendously in recent years, but the increase has not been proportionate to the combination of offsetting factors—increased registrations, increased travel, and increases in construction and maintenance costs. The lack of sufficient revenue is partially caused by the inflexibility of the highway-user tax structure. To be sure, revenues increase as registrations and travel increase and gasoline taxes have been raised in a number of States; but the increase in revenues has not kept pace with the increase in highway costs. For example, from 1940 to 1950 gasoline-tax rates increased only 12.7 percent; yet during the same period unit prices for road construction increased 97 percent and those for maintenance 87 percent. A drastic increase in road-user taxes is not necessarily the best remedy. Many highway administrators are finding that resort to credit financing, together with the assurance of highway revenues adequate to service the debt and meet other highway needs, offers an advantageous solution to their most pressing highway problems.

Credit Financing of Motor Vehicles and Highways Compared

A rather interesting, although unorthodox, comparison can be made between highway bond financing and the use of installment credit to finance the rolling stock of our highway transportation system. The value of new and used vehicles sold in 1948 exceeded $14 billion, of which new and used passenger cars accounted for more than $12 billion. It is reported that 39 percent of all new and used car sales in 1948 involved credit transactions, and that the volume of installment credit was nearly $4.3 billion. In 1950 the volume of installment credit was $7.3 billion. In table 4 and figure 3 the amount of installment credit extended on the sales of new and used passenger cars and the amount outstanding at the end of the year are compared with the total State, county, and municipal long-term highway obligations issued and outstanding, for the postwar years 1946–50.

Although the comparison of short-term installment credit with long-term highway debt

Table 4.—Total highway bonds issued and outstanding, and installment credit on passenger cars extended and outstanding, 1946–50

Year	Installment credit extended on new and used passenger cars during year [1]	Total highway bonds issued during year	Installment credit outstanding at end of year on new and used passenger cars [1]	Total highway debt outstanding at end of year
	Million dollars	*Million dollars*	*Million dollars*	*Million dollars*
1946	1,535	168	878	3,615
1947	2,951	308	1,733	3,655
1948	4,277	475	2,757	3,899
1949	6,031	531	4,129	4,215
1950	7,310	704	5,440	4,557

[1] Board of Governors of the Federal Reserve System, quoted in Automobile Facts and Figures, 1951.

Table 5.—The *Bond Buyer's* index of the municipal bond market (20 bonds)

Year	Index of yield [1]	Year	Index of yield [1]
1900	3.25	1926	4.23
1901	3.10	1927	4.13
1902	3.18	1928	3.87
1903	3.30	1929	4.17
1904	3.40	1930	4.23
1905	3.48		
		1931	4.12
1906	3.53	1932	4.87
1907	3.67	1933	4.61
1908	3.87	1934	5.48
1909	3.76	1935	3.81
1910	3.91		
		1936	3.25
1911	3.98	1937	2.62
1912	4.01	1938	3.16
1913	4.45	1939	2.78
1914	4.16	1940	2.59
1915	4.26		
		1941	2.14
1916	4.08	1942	2.24
1917	4.62	1943	2.17
1918	3.92	1944	1.77
1919	4.44	1945	1.62
1920	4.56		
		1946	1.42
1921	5.06	1947	1.85
1922	4.38	1948	2.36
1923	4.16	1949	2.19
1924	4.37	1950	2.07
1925	4.16	1951	1.66

[1] The yield is the average for the year for the years 1904–14, as of the first of January for the years 1915–45, and as of the first Thursday of January for the years 1946–51.

is somewhat defective, it does point up the tremendous relative volume of financial activity in the passenger-car installment-credit field. It is also notable that installment debt outstanding has increased rapidly in comparison with the more moderate increase in highway debt, with the result that installment credit outstanding exceeded highway debt by nearly $1 billion at the end of 1950. These facts lead to the reflection that if owners of passenger cars have the financial ability and willingness to contract and retire such a tremendous volume of short-term obligations for vehicles, then the retirement of obligations incurred for highways, spread over a number of years, should not prove difficult.

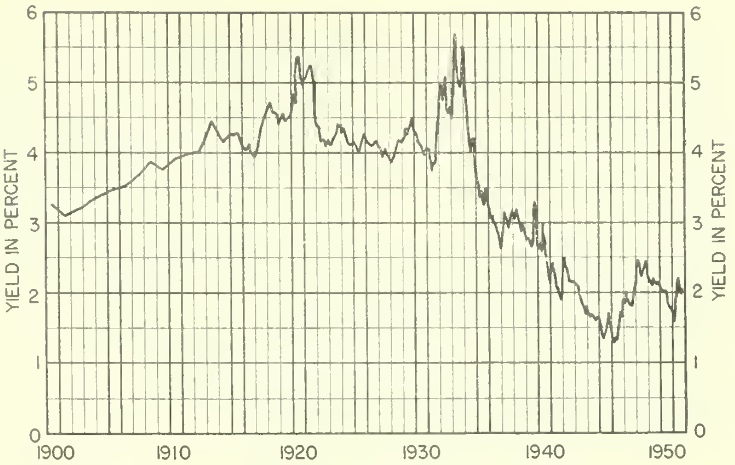


Figure 4.—The Bond Buyer's index of the municipal bond market.

Interest Rates

Low interest rates in recent years have made it possible for the State and local governments to borrow at a considerably less interest cost than was possible in the early years of highway development. In general, the price which a government must pay in the form of interest to induce investors to loan funds includes not only the pure interest cost for the use of the money but also an additional charge for insurance, or reimbursement for risk.

In addition, the price will be affected by the relation existing at any given time between the amount of money available for investment in such loans and the amount sought by potential borrowers, and by the income-tax position of the securities offered. The confidence in the security of State and local highway bonds that has been built up in the minds of investors during the last 50 years has, by reducing the increment of interest charged for risk, contributed to the reduction in total interest costs. The fact that interest on State and municipal bonds is exempt from taxation under the Federal income-tax law, with the high rates on large incomes that have prevailed in recent years, has also been a contributing factor. However, the reduction in pure interest demanded by investors for the use of money has undoubtedly been the most important influence.

Values of *The Bond Buyer's* index of the municipal bond market are given in table 5 and shown graphically in figure 4. This index of calculated yields is the result of averaging the market value, expressed in "basis," of general obligation bonds of selected States and incorporated places. Changes in the bonds used in compiling the index are made from time to time because of trading inactivity in certain included bonds and in order to make the index more truly representative of the municipal market generally.

According to *The Bond Buyer* (22), specific issues are not used in compiling the index. Basis prices used are those applicable to a bond having a maturity of about 20 years and selling at a price close to par, with few exceptions. The average rating of the 20 bonds used in the index falls midway between the four top groups as classified by recognized rating agencies.

While the index is given by years in table 5, it will be noted in figure 4 that annual values are plotted for 1901–14, quarterly values for 1915 and 1916, and monthly values for 1917–51.

Table 6.—Gross highway debt outstanding, 1945–50 [1]

Year	State	Counties and other local rural units	Urban places	Total
	Million dollars	*Million dollars*	*Million dollars*	*Million dollars*
1945	1,638	869	1,223	3,730
1946	1,571	837	1,207	3,615
1947	1,537	864	1,254	3,655
1948	1,735	866	1,298	3,899
1949	1,928	885	1,402	4,215
1950	2,141	904	1,512	4,557

[1] The values given include the debt of highway toll authorities.

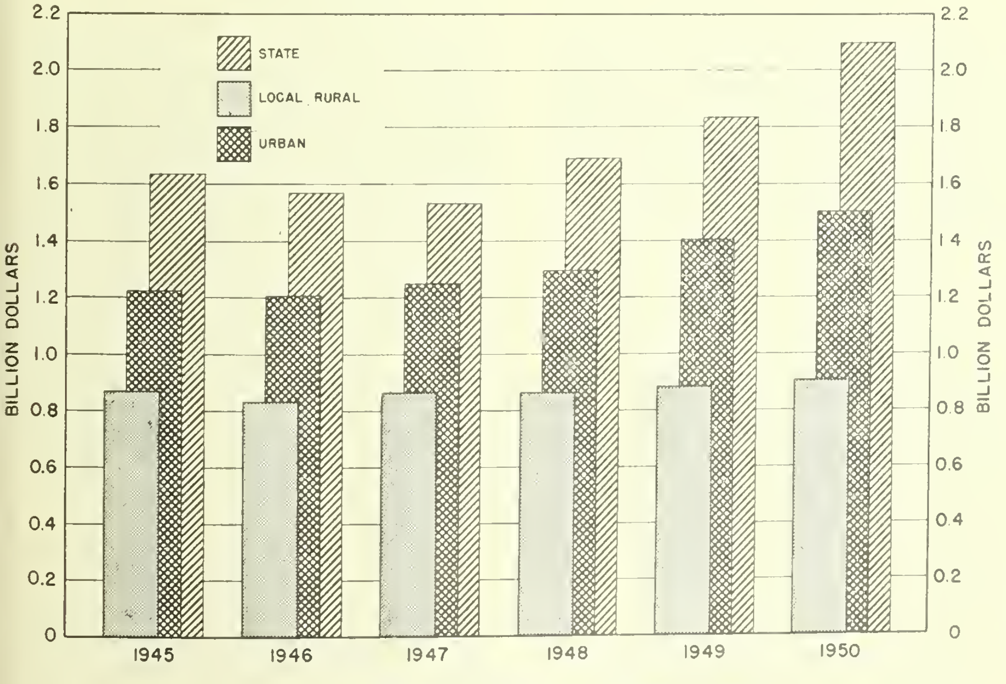


Figure 5.—Gross highway debt outstanding, 1945–50.

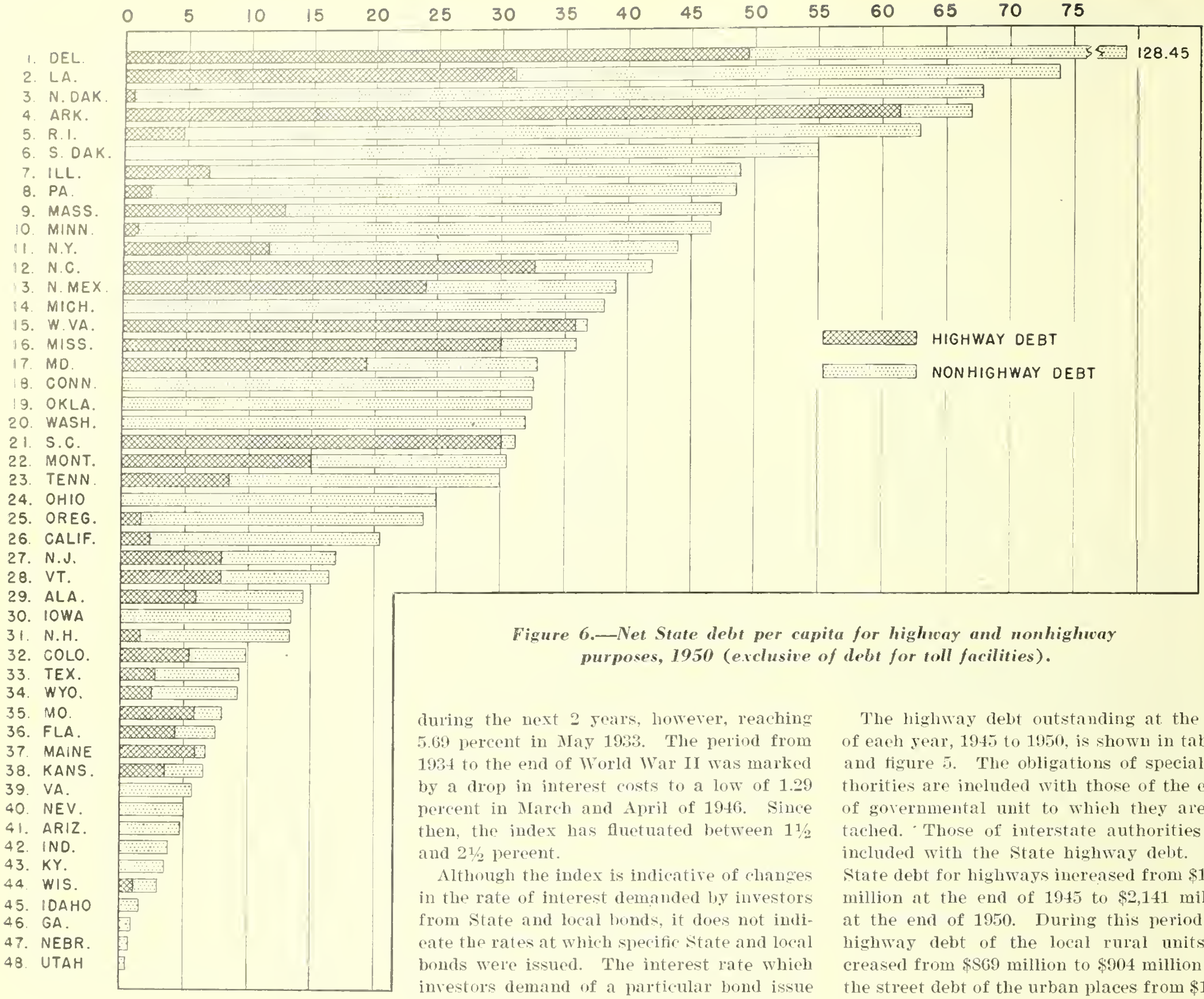


Figure 6.—Net State debt per capita for highway and nonhighway purposes, 1950 (exclusive of debt for toll facilities).

Since the index is indicative of municipal credit in general, it therefore reflects not only changes in interest cost resulting from changes in the volume of money available for investment, the supply of investment opportunities, and other economic conditions; it also reflects changes in conditions peculiar to municipal securities, such as Federal income-tax rates and the faith of investors in the security of municipal bonds as compared with other forms of investment.

As is apparent from figure 4, the interest return on municipal bonds has varied between rather wide limits during the past 50 years. The first 20 years of this century witnessed a gradual increase in interest costs except for a moderate drop during World War I. After climbing to about 5¼ percent in 1920 and 1921, the index rapidly dropped to a little over 4 percent in 1922 and fluctuated near this level during the next 7 years. Following an increase to almost 4½ percent in October 1929, the index dropped to 3.74 percent in June 1931, the lowest since 1907. Interest costs, as represented by the index, increased rapidly during the next 2 years, however, reaching 5.69 percent in May 1933. The period from 1934 to the end of World War II was marked by a drop in interest costs to a low of 1.29 percent in March and April of 1946. Since then, the index has fluctuated between 1½ and 2½ percent.

Although the index is indicative of changes in the rate of interest demanded by investors from State and local bonds, it does not indicate the rates at which specific State and local bonds were issued. The interest rate which investors demand of a particular bond issue may be either higher or lower than the going rate as indicated by the index. Chief among the factors which may contribute to this variation are the credit rating of the issuing government, the security pledged to the redemption of the loan, and the maturity and redemption terms of the issue. All other conditions being equal, the interest rate that a government must pay depends on market conditions at the time of the sale.

POSTWAR CREDIT FINANCING

Since 1945, State and local borrowing for the construction of highways has increased tremendously. During the 5-year period 1946–50, the States and special State authorities or commissions borrowed $1,057 million, the counties and other local rural road agencies $444 million, and the urban units $685 million—a total of $2,186 million. During this period, however, debt incurred in prior years was being retired, with the result that the total outstanding debt increased only $827 million, from $3,730 million at the end of 1945 to $4,557 million at the end of 1950.

The highway debt outstanding at the end of each year, 1945 to 1950, is shown in table 6 and figure 5. The obligations of special authorities are included with those of the class of governmental unit to which they are attached. Those of interstate authorities are included with the State highway debt. The State debt for highways increased from $1,638 million at the end of 1945 to $2,141 million at the end of 1950. During this period the highway debt of the local rural units increased from $869 million to $904 million and the street debt of the urban places from $1,223 million to $1,512 million.

For the country as a whole, the total highway debt outstanding at the end of 1950 is not of alarming proportions. It is slightly more than the $4,532 million spent for all highway capital outlays during the 2 years 1949 and 1950. This includes Federal expenditures of $103 million not classified by system. The outstanding State highway debt is only 72 percent of the total expenditures ($2,931 million) for capital outlays on State highways during those years, and the outstanding county debt for highways is 1.2 times the expenditures ($762 million) for capital outlays on county and other local rural roads during the same 2 years. The outstanding highway debt of the urban places is slightly more than double the sum of the 1949 and 1950 expenditures ($736 million) for capital outlays on city streets.

Per-Capita State Debt

The amount that a State or local unit can borrow for highways, aside from legal restrictions, is conditioned to a considerable degree by the amount of highway debt already in-

Table 7.—Total urban borrowings for highways and streets in selected years

State	1941	1945	Postwar years					
			1946	1947	1948	1949	1950 [1]	Totals, 1946–50
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	359	---	------	---	1,899	5,094	1,528	8,521
Arizona	85	9	9	140	1,179	2,708	830	4,866
Arkansas	21	---	------	35	------	359	200	594
California	113	2.250	------	4,663	3,235	------	8,895	16,793
Colorado	48	35	72	86	86	18	3,700	3,962
Connecticut	870	------	803	544	490	1,099	420	3,356
Delaware	---	------	------	300	----	-	492	792
Florida	-	------	25	455	848	6.135	3,480	10.943
Georgia	100	30	1,694	3.087	757	651	559	6,748
Idaho		------	647	783	127	466	----	2,023
Illinois	70	162	319	2,001	16.560	4.899	8,290	32,069
Indiana	6	12	12	25	31	301	683	1,052
Iowa	3.714	113	550	378	1.276	1,585	1,419	5.208
Kansas	1,442	331	2,640	3.008	6,684	9,266	6,273	27,871
Kentucky	167	------	11	179	98	17	30	335
Louisiana	980	134	372	3,593	5.235	10.965	9,467	29,632
Maine	335	38	230	61	111	148	------	550
Maryland	110	100	------	15.400	100	275	------	15,775
Massachusetts	1,780	------	4.587	5.834	4.572	7.006	8,449	30,448
Michigan	526	217	718	1,121	1.874	7,541	7,159	18,413
Minnesota	1,026	24	1,338	1,116	4,836	3.481	4,984	15,755
Mississippi	503	519	957	3.960	2,921	2.266	4,121	14,225
Missouri	74	178	660	604	900	152	5,857	8,173
Montana	96	19	6	11	195	245	231	688
Nebraska	623	297	170	1.504	3,614	3,028	2,700	11,016
Nevada	15	19	43	196	192	578	182	1,191
New Hampshire	355	------	----	540	693	666	110	2,009
New Jersey	1,081	93	1,835	1,364	1,532	4.700	2,554	11,985
New Mexico	-----	100	25	95	635	552	650	1.957
New York	5,506	14,724	14,413	31,735	21,009	43,009	26,657	136,823
North Carolina	458	172	2,815	3,150	2,983	2,167	2,111	13,226
North Dakota	97	1	166	947	524	1,230	1,056	3,923
Ohio	3.236	3,855	5,757	5,753	8,852	12,753	16,385	49.500
Oklahoma	1,174	135	862	74	229	412	10,278	11.855
Oregon	547	------	85	417	211	592	749	2,054
Pennsylvania	919	78	82	2.535	2.313	12,855	15,061	32,846
Rhode Island	223	478	288	659	837	1.425	3,750	6,959
South Carolina	200	------	------	------	216	728	1,450	2,394
South Dakota	28	15	300	700	75	435	155	1.665
Tennessee	1.043	579	2,099	5,276	1,137	4,226	3,267	16,005
Texas	2.035	3,958	16.382	12,808	15,033	19.537	26,278	90,038
Utah	62	42	6	60	192	44	------	302
Vermont	50	------	200	75	62	101	453	891
Virginia	2,136	------	------	5,122	7.204	2,273	7,075	21,674
Washington	5	595	------	95	18	786	1,000	1.899
West Virginia	2,870	750	20	----	39	85	190	334
Wisconsin	244	------	1,865	1,730	300	1,243	716	5,854
Wyoming [2]	200	---	----	-----	----	-----	-----	--------
Total	35,585	30.062	63,063	122,219	121,914	178,102	199,894	685,192

[1] Estimated. [2] Data not available subsequent to 1941.

curred and also by the amount of the total debt, highway and nonhighway. It can be observed in figure 6 that there is great variance among the States in per-capita debt and its division between highway and nonhighway components. Toll-facility debt is not considered in this part of the discussion. Although the inclusion of their debt as a part of total State debt can readily be justified, the fact that a number of the larger toll facilities draw chiefly from interstate or out-of-State traffic would tend to distort a comparison of per-capita debt which included them. Limited obligations and reimbursement debt are taken into account.

All States have some net debt, but in nine States per-capita total debt does not exceed $5. Sixteen States have no highway debt other than toll-revenue bonds. On the other hand, six States have total per-capita debt of $50 or over, and one, Delaware, has a per-capita debt of $128. The State having the highest per-capita highway debt is Arkansas, with $61. In no other State is per-capita highway debt over $50, although Delaware, Louisiana, North Carolina, West Virginia, Mississippi, and South Carolina have per-capita highway debt between $25 and $50. Some States with little or no highway debt have relatively high nonhighway debts—among them North Dakota, Rhode Island, South Dakota, Minnesota, Connecticut, Oklahoma, Washington, and Ohio. State debt as a whole, highway and nonhighway, is not particularly burdensome in most States, however. In half the States per-capita total debt is less than $25, and in two-thirds is less than $35. Of the total State debt in all States, including limited obligations and reimbursement debt, but not including toll-revenue bonds, highway obligations constitute 30 percent.

Similar comparisons of per-capita debt could be made for the local rural and urban governments (including the counties). However, comparisons of local debt on a Statewide basis would have little meaning, and comparisons among the more than 34,000 local units, many of which are overlapping, which have some highway responsibility would be too voluminous for inclusion here. In attempting to appraise the burden of existing debt, or the capacity for supporting additional debt, consideration must be given not only to the resources and obligations of the unit itself but also to those of overlapping or superior units.

In the paragraphs which follow, more detailed consideration is given to recent trends in highway bond financing at all levels of government. First, attention is given to highway and street obligations issued by the cities and other urban places; and, next, to those of the counties and local rural highway agencies. Finally, the State highway obligations, already touched upon, are subjected to further scrutiny.

Urban Borrowings

Table 7 gives the amounts of highway and street bonds issued by the cities of each State in the years 1946–50, with the years 1941 and 1945 included for comparative purposes. Figure 7 shows the geographical distribution of the total sales for the 5-year period 1946–50.

It is safe to assume that the estimated borrowings of the urban units for street purposes are understated. Although urban street-finance studies are now being conducted in over 40 States, the degree of coverage varies. To complicate matters further, data concerning urban special-assessment districts are not always available and in some States these districts account for a considerable portion of the local street construction.

The urban units of all States but one are known to have issued bonds for street purposes during the period studied; no data were available for Wyoming.

A combination of factors in recent years has encouraged the urban units to enter the credit-finance field to provide funds for city street improvements. Among these factors are the following:

1. The urban units are enjoying their most favorable financial condition since the 1920's. The result is improved credit ratings and lower interest rates.
2. The deficiencies of the urban arterial systems have reached such alarming proportions that immediate action is imperative.
3. State highway-user revenues have reached new heights and the States are assuming their share of the cost of urban projects, thereby easing the burden on the urban units, and making feasible the development of urban arterial facilities.
4. The Federal-aid Highway Act of 1944 specifically authorized $125,000,000 annually for the first three postwar years for the improvement of the urban Federal-aid system; and similar authorizations have been continued in the Federal-aid highway acts of 1948, 1950, and 1952.

Cooperative projects, involving Federal, State, county, and city participation, grow in importance yearly and have given a much-needed impetus to the construction and im-

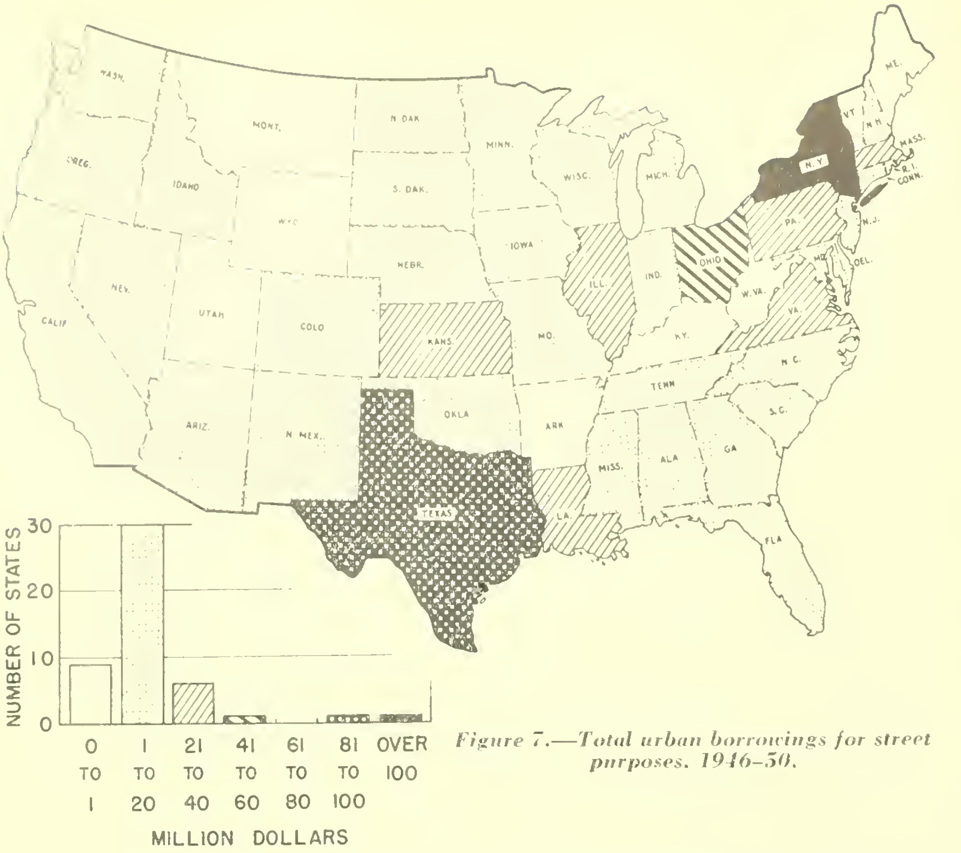


Figure 7.—Total urban borrowings for street purposes. 1946–50.

provement of urban arterial highway facilities. Before the advent of these cooperative projects many of the cities were unable to finance the construction of vital arterials because of the tremendous costs involved. The division of the project costs between themselves and other governmental units has made it possible for the urban units to finance successfully their portion of the projects. In many of the larger urban projects which are being constructed on a cooperative basis the municipal portion of the costs is being financed with the proceeds from bond sales. The methods of financing these projects are discussed in detail later.

In nine States, during the period 1946–50, the urban units borrowed less than $1 million; in thirty States, from $1 to $20 million; and in six, from $21 to $40 million. In Ohio, $49.5 million was borrowed; in Texas, $90 million; and in New York, $136.8 million. The urban units in these three States accounted for 40 percent of the total postwar borrowings of the incorporated and other urban places. Because of its large urban population, it is not surprising that New York State leads the list; but the Triborough Bridge and Tunnel Authority accounted for $81 million of its total.

Credit financing is being widely used by Texas cities, both large and small. About one-fifth of the $90 million borrowed by the urban places in Texas from 1946 through 1950 was for urban expressway construction in cooperation with the State highway department. During the 5-year period, the cities borrowed $18 million for this purpose—$16 million for right-of-way and $2 million for construction. Borrowing for expressways accounted for a significant part of the urban borrowings in other States, among them Georgia and Illinois.

Borrowing by Counties and Other Local Rural Units

Although borrowing by the counties, towns, and townships for highway purposes has been substantial in recent years, it has been relatively small when compared to the scale of credit financing which these same units practiced during the period 1919–29. During this earlier period, however, the local units not only financed the construction of local rural roads but also the early construction of a large portion of the present State highway systems.

It should be remembered that in earlier years the property tax was the main source of revenue for the local rural units, and that it was shortly after 1920 that farm prosperity started its long decline, culminating in a very serious depression by 1930. The counties and towns of today do not rely very heavily on the property tax. Although it is still a substantial source of revenue for local roads, it is no longer the chief source, being exceeded by State aids from road-user taxes in most States.

However, the $444 million borrowed by the local rural units from 1946 through 1950 is no trifling sum, and is indicative of the efforts being made by those units to modernize their road systems. Table 8 shows the borrowings for road purposes by the local rural units in each State for the years 1946–50. The years 1941 and 1945 are included for comparison.

Figure 8 indicates the geographical distribution of borrowings by the counties and other local rural units for roads during the 5-year period. There is no particular pattern evidenced nor are any pronounced regional variations apparent. Borrowings totaling $238 million were reported in 11 out of 16 Southern States; issues totaling $187 million were reported in 18 out of 21 Northern States; but only $13 million was reported borrowed in 7 out of 11 Western States.

Of the 14 States in which the local rural units borrowed less than $1 million during the period, 8 borrowed less than $500,000 and 4 of these less than $100,000.

Ten States incurred no local rural road debt whatsoever. Arkansas local rural units are constitutionally prohibited from incurring debt for highway purposes. In two States, North Carolina and West Virginia, the local rural units exercise no highway function except for the servicing of road debt incurred many years ago. Virginia has only three counties participating in highway activities. The Delaware counties issue bonds only for roads in suburban areas as provided by the 1945 Suburban Road Act.

Eight States accounted for 80 percent of the total local rural borrowings for the 5-year period, with the Texas counties alone issuing approximately 31 percent. That State, in recent years, has experienced tremendous gains in population and industry and, in an effort to keep pace with this expanding economy, the counties have resorted to credit financing for highway construction on a large scale. Of the remaining seven States in which the bulk of the credit financing of highways by local rural units has taken place, borrowing in Illinois and Georgia has stemmed largely from participation of the counties in large expressway projects. In Alabama, Mississippi, New Jersey, New York, and Pennsylvania the local rural units have traditionally borrowed to finance capital outlays.

Possibly another reason, although not so tangible, for the recent increase in credit financing was the material improvement in the credit ratings of the local rural units that has taken place in the last few years. At the end of 1930 the outstanding gross highway debt of these units was estimated at $1,830 million. This had been reduced to $1,212 million by the end of 1941, and to $869 million by the end of 1945. This reduction, coupled with the increased and more stable sources of income, have made the bond proposals of the local units very attractive and the units, for the most part, have obtained favorable interest rates. All of these factors have tended to put a "new look" on the credit financing picture insofar as the counties and other local rural units have been concerned.

Types of Obligations Issued by Local Governments

Table 9 shows the types of local government highway debt issued, redeemed and outstanding for the 5-year period 1946–50. For the urban units, toll-revenue bond issues exceeded retirements by $67 million, and general-obligation and special-assessment bonds showed a

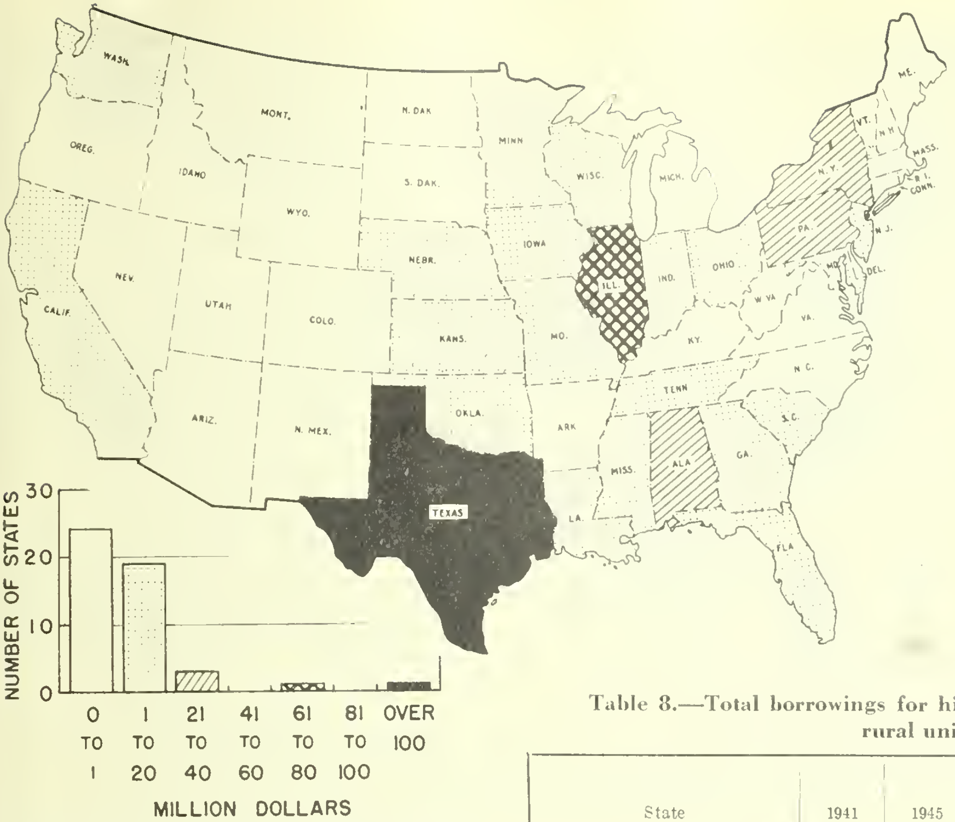


Figure 8.—Total county and local rural borrowings for highway purposes, 1946–50.

net increase of $213 million. The majority of the urban toll-revenue bonds were issued by the Triborough Bridge and Tunnel Authority for the completion of the Brooklyn-Battery Tunnel, by the city of New York in connection with the municipal ferries, and by the city of East St. Louis, Ill., for the Mississippi River bridge: These three units account for over 80 percent of the total toll-revenue bonds outstanding at the end of 1950 with the Triborough Authority accounting for about 72 percent.

General-obligation bonds backed by the full faith and credit of the issuing unit predominate in the urban bond issues. It is believed that special-assessment bonds, once very popular and widely used before the depression, are again assuming some importance in the debt structure of the urban units. However, information on this type of debt is difficult to obtain and the available data are far from complete.

Bonds issued by the counties and local rural units are customarily general obligations backed by the full faith and credit of the issuing authority. Special-assessment bonds and toll-revenue bonds are exceptions. Although the road districts and townships in some States have issued special-assessment bonds in the past, this type of bond has never been as widely adopted by the rural units as by the urban places and has been used but little in recent years.

As shown in table 9, toll-revenue bonds secured only by a pledge of the earnings of the toll facility accounted for $33 million, or about 7½ percent, of the $444 million borrowed by the counties and local rural units from 1946 through 1950. This includes $5 million advanced by the State of California to the Golden Gate Bridge and Highway District for the construction of a new Sausalito approach to the Golden Gate Bridge. The advance is to be repaid when the bonds issued to build the bridge have been retired. Also included is a $7-million Warren County, Miss., issue for the purchase of a privately owned highway and railroad bridge across the Mississippi River at Vicksburg.

Table 9 does not indicate a pronounced shift to toll-revenue financing on the part of the local rural and urban units, such as has marked recent State borrowing policy. This may be accounted for, in part, by the lesser opportunities for toll-facility financing available to the local units. Few roads for which the counties and local rural units are responsible carry sufficient traffic to permit toll financing, and the collection of tolls on city streets is not practical.

Table 8.—Total borrowings for highway purposes by counties and other local rural units in selected years

State	1941	1945	Postwar years					
			1946	1947	1948	1949	1950 [1]	Totals, 1946–50
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	2,330	307	5,278	5,067	7,238	3,772	735	22,090
Arizona	------	------	------	141	222	71	75	509
Arkansas	------	------	------	------	------	------	------	--------
California	68	------	5,000	------	------	------	------	5,000
Colorado	------	------	------	------	------	------	------	--------
Connecticut	42	------	------	------	------	22	------	22
Delaware	------	------	------	135	46	27	25	233
Florida	1,070	14	------	1,500	680	2,450	1,680	6,310
Georgia	255	1,000	1,400	13,300	1,400	3,165	1,200	20,465
Idaho	------	------	------	------	------	------	------	--------
Illinois	1,262	6,769	4,193	7,750	4,185	27,120	25,510	68,758
Indiana	------	60	80	60	542	1,954	867	3,503
Iowa	290	236	453	754	1,341	1,747	------	4,295
Kansas	1,552	20	510	2,152	627	151	250	3,690
Kentucky	60	41	------	40	32	16	------	88
Louisiana	1,384	60	758	199	100	436	714	2,207
Maine	------	3	27	44	60	402	150	683
Maryland	------	------	------	1,500	------	2,500	794	4,794
Massachusetts	------	8	83	89	221	250	200	843
Michigan	------	------	------	96	156	80	100	432
Minnesota	713	246	645	1,469	562	817	750	4,243
Mississippi	509	662	893	8,776	2,929	2,248	3,212	18,058
Missouri	12	40	2,761	1,810	1,034	1,100	1,000	7,705
Montana	------	------	200	------	------	------	54	254
Nebraska	------	------	------	91	------	------	1,970	2,061
Nevada	------	------	15	------	------	------	------	15
New Hampshire	80	------	------	149	142	150	100	541
New Jersey	2,465	553	2,185	4,331	3,830	5,945	3,507	19,798
New Mexico	------	------	------	------	------	------	------	--------
New York	2,086	594	680	6,898	4,395	3,707	13,005	28,685
North Carolina	------	------	------	------	------	------	------	--------
North Dakota	25	80	6	252	111	34	50	453
Ohio	925	------	725	2,100	62	644	291	3,822
Oklahoma	------	------	3,400	3,388	100	9	900	7,797
Oregon	------	------	------	450	------	150	------	600
Pennsylvania	4,049	892	2,665	3,619	8,937	10,690	10,390	36,301
Rhode Island	------	------	------	------	------	------	------	--------
South Carolina	356	------	1,575	2,170	------	1,700	1,000	6,445
South Dakota	71	------	------	8	282	------	250	540
Tennessee	840	460	1,062	2,151	2,035	3,170	1,530	9,948
Texas	10,293	10,140	15,292	29,992	42,206	21,000	30,893	139,383
Utah	6	------	60	------	8	------	------	68
Vermont	------	------	------	------	------	------	------	--------
Virginia	------	------	------	------	------	------	------	--------
Washington	------	------	------	------	1,500	3,000	1,700	6,200
West Virginia	------	------	------	------	------	------	------	--------
Wisconsin	678	------	390	5,119	590	400	660	7,159
Wyoming	------	------	------	------	------	------	------	--------
Total	31,401	22,185	50,336	105,600	85,573	98,927	103,562	443,998

[1] Estimated.

Table 9.—Types of local government highway debt issued, redeemed, and outstanding, 1946–50 [1]

Year	Issued: General obligation and special assessment bonds	Issued: Toll-revenue bonds	Redeemed: General obligation and special assessment bonds	Redeemed: Toll-revenue bonds	Outstanding at end of year: General obligation and special assessment bonds	Outstanding at end of year: Toll-revenue bonds
LOCAL URBAN GOVERNMENTS						
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
1945					985,032	237,837
1946	49,537	13,526	74,439	4,469	960,041	246,894
1947	102,399	19,820	68,051	6,682	994,306	260,032
1948	95,368	26,546	70,540	6,917	1,018,238	279,661
1949	135,667	42,435	71,995	12,153	1,092,514	309,943
1950	188,477	11,417	73,747	16,253	1,207,350	305,107
Total	571,448	113,744	358,772	46,474		
COUNTY AND LOCAL RURAL GOVERNMENTS						
1945					816,345	52,773
1946	41,607	7,370	76,756	844	777,570	59,299
1947	95,232	10,965	78,380	1,369	794,987	69,006
1948	84,291	1,535	75,562	2,680	798,066	67,861
1949	94,033	4,526	77,467	2,922	815,019	69,465
1950	96,072	8,270	81,886	3,114	829,205	74,621
Total	411,235	32,666	390,051	10,929		

[1] Refunding bonds not included. As adjustments to outstanding debt made possible by more complete information could not always be carried back through prior years, the computed outstanding debt will not always agree with the actual outstanding debt.

State Borrowings

Table 10 gives the borrowings for highway purposes by the various States for the 5 years 1946–50. Again, the years 1941 and 1945 have been included for comparative purposes. Figure 9 shows the geographic distribution of the total borrowings for highways during the 5-year period covered. The geographic distribution is not highly significant but it does indicate a concentration of borrowings along the eastern seaboard. This is not surprising, as construction in urban and industrial areas is an expensive process.

Twenty-five States accounted for approximately 99 percent of all the bonds issued, and five States—Maryland, Massachusetts, New Jersey, North Carolina, and Pennsylvania—issued 55 percent of the total. The Atlantic seaboard States accounted for a little over 80 percent of the total issued.

The two largest borrowers used bond receipts for almost diametrically opposite types of roads. The $125-million North Carolina bonds were issued to improve former county roads now under State control, while Pennsylvania issued $164.5 million to extend the Pennsylvania Turnpike, $15 million for State road and bridge construction, and $8.5 million to purchase privately owned toll bridges.

During the 5-year period, 20 States incurred no highway debt whatsoever. Three States borrowed less than $1 million: Missouri and Texas assumed small amounts of reimbursement debt, and New Hampshire issued $650,000 in toll-bridge revenue bonds. (The bonds which replaced notes issued to finance construction of the New Hampshire Turnpike were not issued until 1951.)

The array of States would be modified somewhat if 1951 issues had been included. The latest available information indicates that more than $460 million in State highway obligations (including the issues of State toll authorities) were sold during that year. Of this total, $186 million were toll-road and toll-bridge bonds and $274 million were toll-free issues of various kinds.

Types of Obligations Issued by States

In 1948, 1949, and 1950, bond issues for State highway construction averaged over $300 million per year. The debt transactions that have taken place during the postwar period, however, have exhibited marked differences from the experience of the 1920's and 1930's, reflecting new developments in highway finance and administration.

The postwar issues may be divided into three groups, depending on the type of security offered and income pledged to service the debt. The three types are general-obligation bonds, limited-obligation bonds, and toll-revenue bonds. The reimbursement obligations described previously, although not evi-

Table 10.—Total State borrowings for highway purposes in selected years [1]

State	1941	1945	Postwar years: 1946	1947	1948	1949	1950	Totals, 1946–50
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama								
Arizona								
Arkansas						7,000	7,000	14,000
California								
Colorado							6,300	6,300
Connecticut				10,000		1,000		11,000
Delaware					42,500	7,000	2,000	51,500
Florida			3,150		10,501	5,720	28,000	47,371
Georgia								
Idaho								
Illinois								
Indiana								
Iowa	218							
Kansas								
Kentucky				1,000				1,000
Louisiana	116		2,960	19,000			10,000	31,900
Maine	500		15,000	5,000		600		20,600
Maryland	5,434	1,560			37,138	26,688	25,000	88,826
Massachusetts					27,006		60,000	87,000
Michigan								
Minnesota								
Mississippi			5,000	10,000	10,000	5,000	2,217	32,217
Missouri		297		21	2			23
Montana			323		5,000	5,500		10,823
Nebraska								
Nevada								
New Hampshire					650			650
New Jersey				7,400	5,000	4,000	75,500	91,900
New Mexico	2,000		2,000	4,000	3,000			9,000
New York	15,000		10,655	8,500			22,000	41,155
North Carolina						50,000	75,000	125,000
North Dakota					2,925			2,925
Ohio								
Oklahoma							31,000	31,000
Oregon								
Pennsylvania			1,500		87,000	101,000		189,500
Rhode Island								
South Carolina	690	4,100	6,000	5,000	20,000	10,000	10,000	51,000
South Dakota								
Tennessee								
Texas	3,143			98	38	4	339	479
Utah								
Vermont							2,800	2,800
Virginia						19,000	23,000	42,000
Washington				3,650	14,000	1,650		19,300
West Virginia	6,350	5,000	8,000	5,000	3,000	9,500	18,500	44,000
Wisconsin	714			1,367	318	400	1,468	3,553
Wyoming								
Total	34,165	10,897	54,528	80,036	268,072	254,062	400,124	1,056,822

[1] Refunding bonds not included.

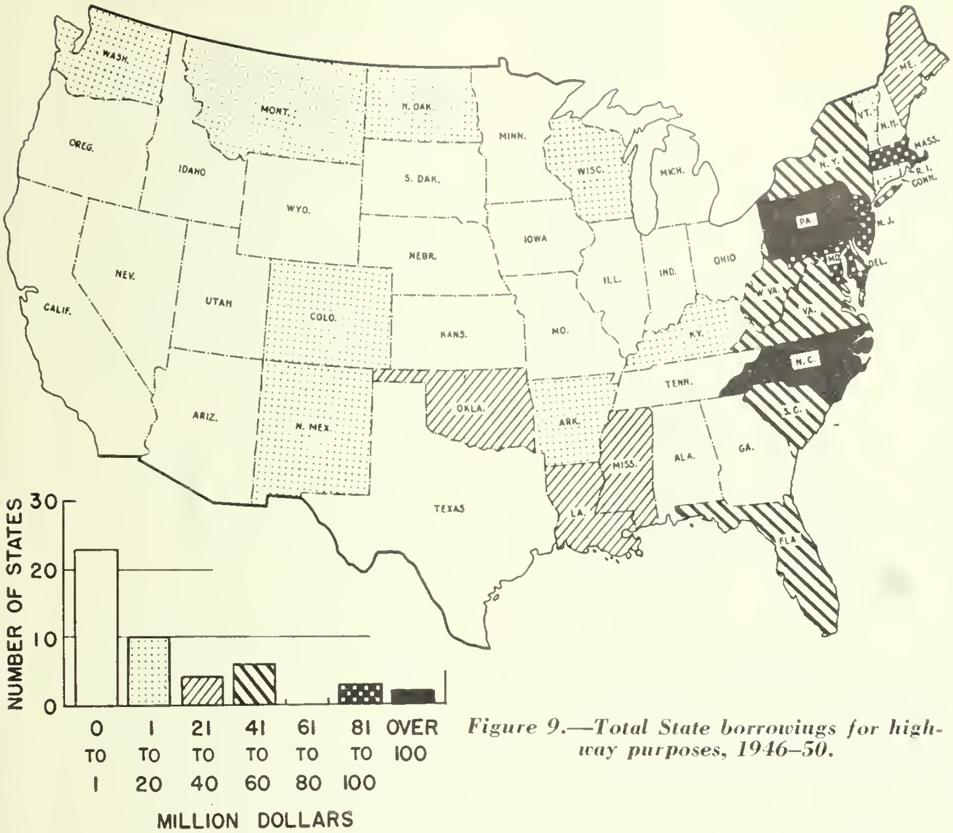


Figure 9.—Total State borrowings for highway purposes, 1946–50.

denced by State bonds, form a fourth type of obligation incurred by the States for highways. Figure 10 and table 11 give issues, redemptions, and amounts outstanding for each type for the period 1946–51.

In spite of a net increase of $163 million in debt outstanding at the end of 1951, general-obligation issues showed little relative net increase in the postwar period because of the large amount (over $1 billion) outstanding at the beginning. General-obligation bonds are secured by the full taxing power of the State but, in the case of highway bonds, are most frequently serviced from highway-user revenue funds. Many States have specifically pledged all or a portion of their road-user revenues in addition to the general taxing power. Alabama, for example, services its first highway bond issue from motor-vehicle license revenues, and its second and third issues from the motor-fuel tax. Louisiana allocates the amount required from the income obtained from 4 cents of a 9-cent gasoline tax for debt service on State highway obligations. In Arkansas, 70 percent of the first $10,250,000 of revenues of the State highway fund is set aside for debt service. In the newer Massachusetts issues, an increase in the gasoline tax of 1.3 cents is dedicated to pay interest and principal on the bonds. In many States it is probable that the pledge of road-user revenues for debt service offers, in effect, as great a security for the bond issues as does the additional pledge of the full faith and credit of the State, since the highway revenues represent one of the larger and more constant sources of the State revenues.

Reimbursement obligations, once forming a relatively large portion of outstanding State highway debt, have now ceased to be a significant factor. About $138 million of such debt was outstanding at the end of 1945; only $52 million was outstanding at the end of 1951.

Some recent bond issues have been secured only by the pledge of certain highway-user revenues. This type of limited obligation has shown a marked net increase since the war, from $97 million on January 1, 1946, to $320 million on December 31, 1951. The large issues in Maryland and Mississippi have been secured only by the highway funds. In Florida the Improvement Commission bonds for State roads are limited State obligations secured by a portion of the gasoline tax. The Pennsylvania Highway and Bridge Authority bonds are also limited obligations backed only by the motor license fund revenues. The most recent instance of limited-obligation financing on a large scale is found in Michigan's Act 22, enacted in 1950. The State highway commissioner, counties, cities, and villages are authorized to cooperate with each other or the Federal Government in issuing limited-obligation bonds for express highways, in an amount, however, not to exceed $200 million. Debt service is to be derived from road-user revenues and other pledged funds. Limited-obligation bonds amounting to $277 million have actually been issued by the States since 1945, an indication of the popularity of this type of borrowing.

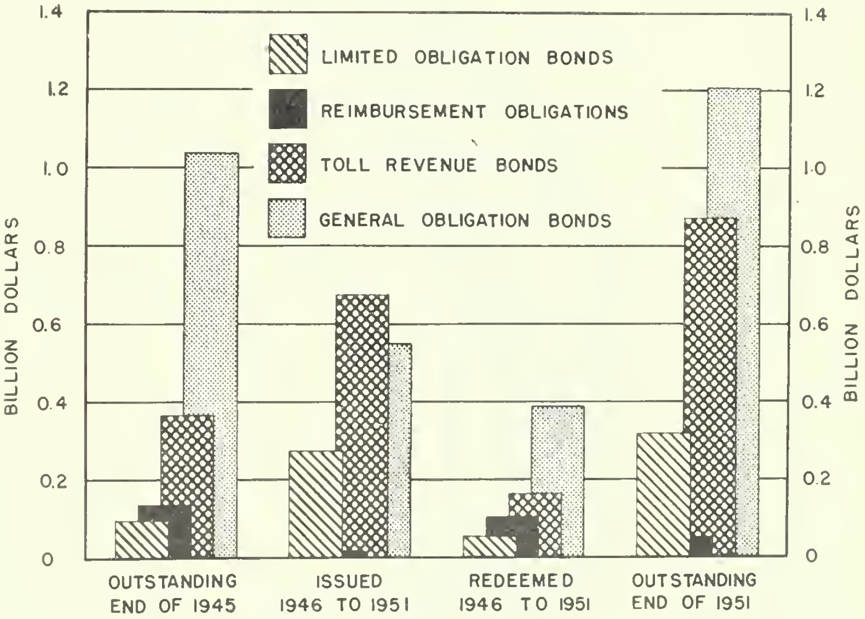


Figure 10.—Types of State highway debt.

Table 11.—Types of State highway obligations issued, redeemed, and outstanding, 1946–51 [1]

Type of obligation	Outstanding Dec. 31, 1945	Issued 1946–51	Redeemed 1946–51	Outstanding Dec. 31, 1951
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
General-obligation bonds	1,039,799	549,246	386,302	1,202,743
Reimbursement obligations	138,324	16,642	103,027	51,939
Limited-obligation bonds	96,570	277,012	53,886	319,696
Toll-revenue bonds	363,123	674,368	165,606	871,885
Total	1,637,816	1,517,268	708,821	2,446,263

[1] Excludes refunding.

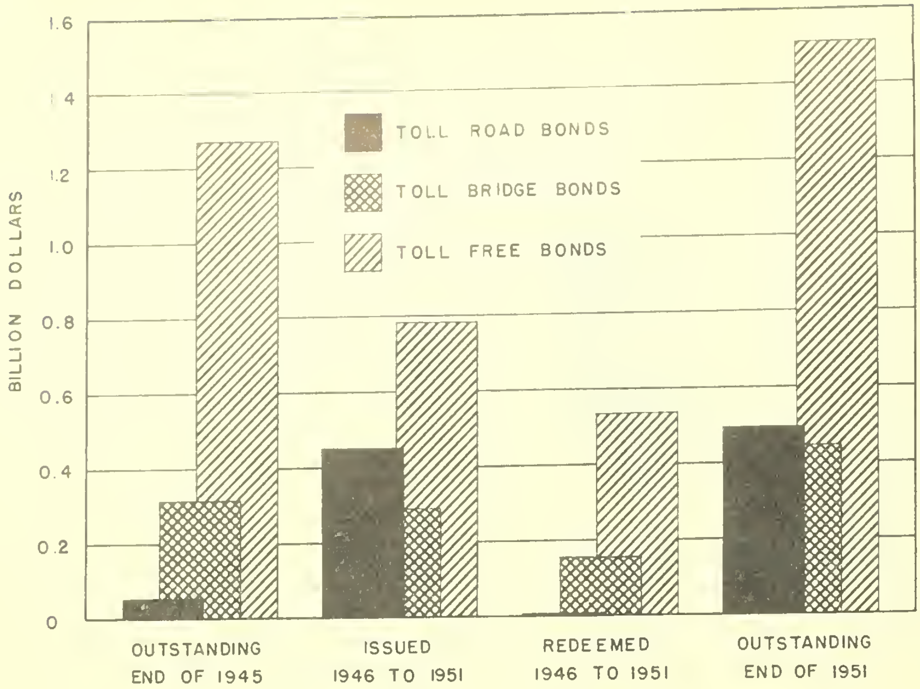


Figure 11.—Purpose of State highway debt.

State Revenue-Bond Financing

Revenue-bond financing for toll crossings and toll roads has been a dominant feature of the postwar bond picture. Toll-revenue issues since 1946, refunding excluded, have totaled over $670 million, and the total outstanding has been more than doubled—from $363 million to $871 million during the 6-year period. None of this financing involves the pledge of any other revenues or the taxing power on the part of the State; it is supported solely by the earning power of the toll facilities. Immense projects such as the Delaware Memorial Bridge, the Chesapeake Bay Bridge, the Pennsylvania Turnpike extensions, the New Jersey Turnpike, and the Mystic River Bridge which have been completed or placed under construction have been financed in this manner. Since this type of debt has reached significant proportions, a brief discussion of the history of revenue financing and public authorities is in order.

The earliest revenue-bond financing by governmental units occurred about 1890 in the State of Washington when, under a constitutional provision, municipalities were permitted to incur debt for the purpose of supplying water, lighting, and sewers. By 1910 revenue bonds were generally used throughout the United States in financing the construction of municipally owned utilities. Water districts in Maine issued revenue bonds during the 1890's, but no other public authorities participated in this type of financing on a large scale until the Port of New York Authority toll-revenue bond issues of 1926. About the same time the Kentucky and Ohio State Bridge Commissions resorted to similar borrowing. In the 1930's revenue-bond financing was further stimulated among the States and local units by the Federal Government's Reconstruction Finance Corporation and Public Works Administration through their policies of bond purchasing and direct grants for self-liquidating projects.

In more recent years toll-revenue financing has been largely undertaken by independent State authorities created for specific purposes. Usually the revenue derived from the facility is the sole pledge for the redemption of the bonds. Notable among such authorities concerned with highway facilities are the California Toll Bridge Authority, Pennsylvania Turnpike Commission, New Jersey Turnpike Authority, and Triborough Bridge and Tunnel Authority.

State highway obligations, which were classified according to the security pledged for their retirement in table 11 and figure 10, are classified according to the purpose for which the debt was incurred in table 12 and figure 11. It will be noted that the total toll-road and toll-bridge bonds reported in table 12 slightly exceed the toll-revenue bonds reported in table 11. This results from the fact that a few toll-facility bonds are general obligations of the State in which they are located.

As can be observed, toll-road bonds show the greatest increase—from $54 million outstanding at the end of 1945 to $491 million outstanding at the end of 1951. As most of the toll-road debt is relatively new, little of it was retired during the 6-year period. Toll-free facility bonds show a net increase in the amount outstanding of about $240 million, and toll-bridge bonds a net increase of $130 million. It is noteworthy that the amount of toll-road debt outstanding ($491 million) now exceeds the toll-crossing debt outstanding ($445 million) for the first time.

It should be pointed out that against the $2.4 billion in gross debt outstanding there is approximately $300 million in debt-service reserves of which about $120 million is to be applied against toll-facility obligations and the remaining $180 million against the toll-free debt outstanding. The debt-service reserves of the Pennsylvania Turnpike Commission and the Port of New York Authority constitute the greater portion of the toll-facility reserves.

Table 12.—Purposes of State highway obligations issued, redeemed, and outstanding 1946–51 [1]

Purpose of issue	Outstanding Dec. 31, 1945	Issued 1946–51	Redeemed 1946–51	Outstanding Dec. 31, 1951
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Toll-free bonds	1,268,557	781,130	538,945	1,510,742
Toll-bridge bonds	314,778	287,638	157,705	444,711
Toll-road bonds	54,481	448,500	12,171	490,810
Total	1,637,816	1,517,268	708,821	2,446,263

[1] Excludes refunding.

Recent Examples of Credit Financing in the Highway Field

The remainder of this report is devoted to case studies of bond-issue financing in individual States. Because of the variety in the kinds of bonds issued, the nature of the security, and the circumstances surrounding the issues, it has been found desirable to segregate and discuss in turn examples of credit financing under the following main headings: Toll bridges and other toll crossings; toll roads; special State authorities; cooperative projects; and regular credit financing of State highways.

TOLL CROSSINGS

The use of toll-revenue bonds to finance the construction of expensive bridges is a time-honored practice. Many bridges that are now free were originally constructed, or purchased from private owners, with the proceeds of revenue bonds that have since been retired from toll collections. The States of Kentucky and Ohio, among others, have provided many bridges in this manner. The Federal Government has recognized the special character of toll bridges by legislative acts which authorize the use of Federal-aid funds in freeing toll bridges on the Federal-aid system, by providing not to exceed 50 percent of the cost; and also the use of Federal funds in the construction of toll bridges, provided that they shall become free when the cost to the State or local government has been retired (23).

Since the toll-revenue method of financing is usually reserved for the more difficult and expensive crossings, it is not surprising that some of the more imposing highway facilities of this type are toll crossings. Among those constructed in the past are the San Francisco-Oakland Bay and Golden Gate Bridges in California, and the George Washington Bridge and the Holland and Lincoln Tunnels connecting New York and New Jersey. Four bridges—the Delaware Memorial, the Tacoma Narrows, the Chesapeake Bay, and Lower Tampa Bay Bridges—are discussed here as illustrating postwar developments in the field of toll crossings.

Delaware Memorial Bridge

The Delaware Memorial Bridge, completed in 1951, was constructed in order to eliminate the New Castle-Pennsville Ferry, a bottleneck to interregional traffic along the Atlantic seaboard. It was undertaken by the State of Delaware which created a special Delaware Crossing Division in its Highway department to handle the construction, financing, and operation. A $40-million revenue-bond issue was sold in June 1948 at a premium of 0.5 percent, with interest at 4 percent and maturity scheduled for 1978, with provision for call and redemption at a premium in 1953 or thereafter. In 1951, a supplemental issue of $3.9 million was sold, with interest at 3¾ percent, to provide funds necessary to complete construction.

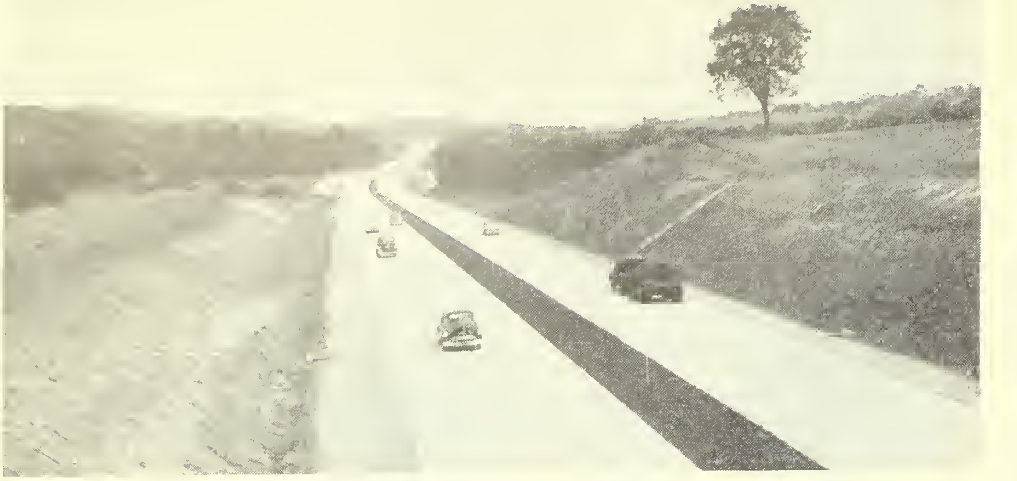
The bridge, carrying four lanes of traffic, has a total length, including approaches, of 10,750 feet and includes a 2,150-foot suspension span. It connects U S 40 and U S 13 in Delaware with important routes in New Jersey, including the New Jersey Turnpike, and thereby effects a direct highway connection between Washington and Baltimore and the New York metropolitan area.

Tacoma Narrows Bridge

The Tacoma Narrows Bridge, originally a two-lane suspension structure, was first completed in 1940; but shortly thereafter the deck structure collapsed due to unusual wind conditions in the area—the famous "Galloping Gertie" episode. Reconstruction was delayed several years as elaborate aerodynamic studies were necessary to perfect a safer bridge design. The new structure utilizes portions of the original piers and towers and carries four traffic lanes. The deck design incorporates unusual features to allow for wind pressures.

Financing of the new bridge was accomplished through the Washington Toll Bridge Authority in March 1948 with the sale of a $14-million revenue-bond issue. The Toll Bridge Authority encountered great difficulty in selling the bonds and, after repeated efforts had failed, it entered into an agreement with Pierce County, Wash., whereby that county

The Chesapeake Bay Bridge, Maryland.



Eastern extension of the Pennsylvania Turnpike.

guaranteed interest payments up to $1.5 million which it derived by issuing its own general-obligation bonds. The $1.5 million was designated as the Tacoma Narrows Toll Bridge county aid fund, and can be drawn upon when toll revenues do not meet interest requirements on the toll-bridge bonds. Even with this guarantee the $14-million revenue issue sold at 95 percent of par with nominal interest at 3¾ percent. The bridge was opened to traffic October 15, 1950. The revenues to date are considerably in excess of the preliminary estimates and, it continued at the present level, will assure the financial success of the bridge.

Chesapeake Bay Bridge

A toll crossing of Chesapeake Bay in Maryland is one of the primary postwar highway projects in that State. Financial arrangements were made in 1948 with the issue of $37.5 million toll-revenue bonds at par. Interest was 3.2 percent on $19 million of term bonds maturing in 1972, and 2½ to 3 percent on $18.5 million of serial bonds. Higher costs encountered during the following year made it necessary to sell an additional $6,425,000 of term bonds bearing interest at 3.2 percent and maturing in 1972. Completion is scheduled for July 1952.

The enabling statute required that outstanding debt of the Chesapeake Bay Ferry System be redeemed before revenue bonds could be issued. Revenues derived from the operation of existing toll bridges on the Potomac and Susquehanna Rivers are to be combined with those of the Chesapeake Bay Bridge for debt service. Of the proceeds of the $37.5 million issue, $362,000 were used to refund the outstanding toll-bridge revenue bonds.

In view of the established earning power of the Susquehanna and Potomac River Bridges, the security prospects of the Chesapeake Bay Bridge bonds are excellent. In fact, during 1949 and 1950, $3,466,000 of excess toll revenues from those two bridges were deposited in the Chesapeake Bay Bridge construction fund. The earnings of the Susquehanna and Potomac River Bridges, now debt free, average over $2 million a year, which is a substantial share of the average annual debt service of $2.8 million on the Chesapeake Bay Bridge bonds. Thus there is in Maryland the beginning of a system of toll-bridge financing, wherein a bridge is not made toll-free when its debt is paid; but instead its continued earnings are used to buttress the security of new enterprises.

Lower Tampa Bay Bridge

The crossing of Lower Tampa Bay in Florida involves a 30-year term issue of $21.5-million revenue bonds by the Florida State Improvement Commission, sold in September 1951. The issue was sold at par and bears interest at 3¾ percent. In connection with this bridge the State Improvement Commission received authority in 1949 to acquire from the St. Petersburg Port Authority the ferry now serving the traffic at the projected bridge crossing. Surplus revenue from the ferry may be applied to the bridge construction fund. Contracts for the bridge were let shortly after the bonds were sold in September 1951. As of November 30, 1951, $11,730,000 in construction contracts had been let. Unlike other State Improvement Commission issues, the Lower Tampa Bay Bridge bonds are not secured by a portion of the State gasoline tax, but only by the earnings of the toll bridge.

TOLL ROADS

The modern toll road is a comparatively recent development. Although numerous turnpikes were constructed during the early nineteenth century, following completion of the Philadelphia–Lancaster Turnpike in 1795, most of these ventures were not financially profitable. As the toll system was abandoned, the turnpikes reverted to public ownership. Turnpikes did not play an important part in the era of highway improvement that followed the advent of the automobile. The modern toll-road movement may be said to have begun with the construction of the Pennsylvania Turnpike in 1938–40.

Toll roads are superficially similar to toll bridges and other toll-crossing facilities in that they are, or purport to be, self-supporting enterprises. However, an important difference is the circumstance that a toll bridge or other crossing is a monopoly or near-monopoly capable of serving all traffic desiring to cross at that point, while a toll road, because of its necessarily limited number of points of access, cannot serve purely local traffic traveling short distances. There is no important difference in the adequacy of service rendered all traffic by a toll bridge and a free bridge. A toll road, however, is a supplementary facility for a special class of traffic and, alone, cannot adequately serve all traffic.

It is not the purpose of this article to discuss the advantages and disadvantages of toll roads. However, a brief discussion of the toll roads that have been built, are under construction, or planned, is appropriate. A complete treatment of the modern toll-road situation will be found in *Toll Roads and the Problem of Highway Modernization*, by Owen and Dearing (*24*).

Pennsylvania Turnpike Commission

Article IX, section 4 of Pennsylvania's State constitution prohibits the State from incurring debt except to meet casual deficits or to repel invasions. Further restrictions limit the amount of debt that can be incurred to meet casual deficits to $1 million. By means of amendments to the constitution, debt in excess of this amount can be incurred; but two consecutive sessions of the legislature must act favorably on the proposed amendment, and it is then submitted to the electorate. A further restriction on amendments is that no particular section of the constitution can be amended more often than once in 5 years.

Despite these difficulties Pennsylvania, in 1918, passed an amendment which authorized the incurrence of debt for highway purposes up to $50 million. Five years later a similar amendment raised this authorization to $100 million. In 1933 another amendment authorized the State to issue bonds up to $10 million for the purchase of privately owned toll bridges which were located wholly within the State, but no bonds were issued for this purpose until 1949.

In spite of precedents for the issuance of general-obligation bonds by constitutional amendment, Pennsylvania has seen fit to create special authorities empowered to issue bonds backed only by the credit of the authority. Among the Pennsylvania authorities created for the purpose of issuing bonds are the Pennsylvania Turnpike Commission, the State Highway and Bridge Authority, and the General State Authority. The latter has not issued any bonds for highway purposes, although highways are within the scope of its

Table 13.—Bond issues of the Pennsylvania Turnpike Commission

Year issued	Term (in years)	Interest rate	Par value	Premium or discount [1]	Accrued interest	Net proceeds
		Percent				
1938	30	3.75	$40,800,000	−$3,368,911	$235,687	$37,666,776
1943	30	3.75	1,500,000	+19,500	23,281	1,542,781
1946 [2]	30	2.50	46,000,000	+431,940	5,431	46,437,371
1948 [2]	20	2.25	47,000,000	−1,034,000	223,250	46,189,250
1948	40	3.25	87,000,000	−1,914,000	536,915	85,622,915
1949	40	2.90	77,500,000	−1,550,000	742,917	76,692,917

[1] Premium shown as plus; discount as minus.

[2] Refunding issues.

authorized activities. Of the three, only the Pennsylvania Turnpike Commission has constructed toll roads. The Pennsylvania Turnpike Commission, an instrumentality of the Commonwealth of Pennsylvania, was created by Act No. 211 of the General Assembly, approved May 31, 1937, Public Law 774, as amended by the Acts of May 24, 1945, Public Law 972, and Feb. 26, 1947, Public Law 17.

The Turnpike Commission consists of five residents of the Commonwealth, one of whom, the Secretary of Highways, is ex-officio a member. The other four members are appointed by the Governor with the advice and consent of two-thirds of the Senate. The Commission is authorized and empowered to construct, operate, and maintain a turnpike through the Allegheny Mountains, and to acquire right-of-way and to construct tunnels and bridges necessary to provide an all-weather, low-grade highway between the eastern and western sections of the Commonwealth. The Commission is further authorized by the act to issue revenue bonds of the Commonwealth, not only for the purpose of paying the cost of the turnpike, but also for the purpose of refunding any bonds then outstanding. Under provisions of the act such bonds shall not be deemed to be a debt of the Commonwealth or a pledge of the faith and credit of the Commonwealth, and the Commonwealth is not directly, indirectly, or contingently obliged to levy or to pledge any form of taxation or to make any appropriation for their payment. When all debt requirements are satisfied the turnpike becomes a part of the system of State highways, free of tolls.

The Pennsylvania Turnpike

The original turnpike constructed under authority of this act extends from Middlesex to Irwin. The construction of this first section of the Pennsylvania Turnpike was financed by the proceeds of a $40.8-million bond issue and a PWA grant of $29.25 million. This section also had the advantage of utilizing millions of dollars' worth of grading and some tunnel work done by the never-finished South Penn Railroad.

The acts of May 16, 1940, Public Law 949, and of June 21, 1947, Public Law 877, authorized the Commission to construct the eastern or Philadelphia extension of the Turnpike and the acts of June 11, 1941, Public Law 101, and of June 21, 1947, Public Law 877, granted it the authority to construct the western extension.

Construction on the first section of the Turnpike began in 1938 and it was opened to traffic October 1, 1940. Work on the eastern extension began in September 1948, and it was opened to traffic in October 1950. The western extension was begun in October 1949 and was opened in December 1951. The Commission now has the authority to construct connecting links north to the New York Thruway, south to the Maryland border, and also to construct connections with the New Jersey Turnpike.

The disinclination of the Commonwealth to pledge its full faith and credit to the revenue bonds, or to issue general-obligation bonds for the construction of the Turnpike, was reflected in the high interest rate the bonds carried and the substantial discounts at which the bonds were sold. Table 13 gives the bond transactions of the Commission from its inception to date.

The 1938 issue of $40.8 million and the 1943 issue of $1.5 million (both due in 1968) were retired in 1946 by a refunding issue of $46 million bearing a 2.50 percent interest rate. Although the issuance of the refunding bonds was presumably dictated for reasons of economy, it also appeared to be a matter of necessity. The Turnpike was opened to commercial traffic just 14 months before the entrance of this country into World War II, and during the war years did not earn sufficient revenue to meet current expenditures, with the result that there was no accumulation of surplus for debt reduction. Since operating funds were depleted to make up the difference between current revenue and current expenditures, it became necessary for the Commission to acquire more capital. Of the extra proceeds ($3.7 million) over the amount necessary for refunding purposes, $1,692,000 was used to pay a call premium of 4 percent on the refunded issue, $1,500,000 was earmarked for construction, and the remaining $508,000, plus $432,000 in premiums, was presumably available for debt service or other purposes.

In 1948, refunding bonds of $47 million bearing an interest rate of 2.25 percent were issued to retire and to pay interest on the $45,086,000 of 2.50's then outstanding. The 2.50's were called on December 1, 1951, at 104, the call premium amounting to almost $2 million.

During 1950 and 1951 interest charges amounted to $7.2 million annually while the gross debt outstanding at the end of 1951 was about $208 million. Interest requirements on the present debt will be about $6 million in 1952 and will gradually decrease thereafter as the debt is extinguished, dropping to about $4 million at the end of 20 years. The debt-service schedule provides for total principal and interest payments of from $9 to $10 million annually until the debt is completely retired in 1988.

The Maine Turnpike

The Maine Turnpike was the first postwar toll facility and consists of 47 miles of highway between Kittery and Portland. It is financed and operated by the Maine Turnpike Authority, a body established by the legislature in 1946 with authority to finance, build, and operate a toll highway from Kittery to Fort Kent on the Canadian boundary in the northern part of the State. There has been some consideration of toll projects north of Portland, but the 47 miles between Kittery and Portland comprise the Maine Turnpike at this time.

The original financing was accomplished through the issuance in 1946 of $15 million

The Maine Turnpike.

Table 14.—Maine Turnpike receipts and expenditures, 1947–50

	1947 [1]	1948	1949	1950	1951
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Receipts:					
Tolls	19	655	842	1,008	1,182
Rental		12	25	29	30
Non-operating income		15	5	4	5
Total	19	682	872	1,041	1,217
Expenditures:					
Operation and maintenance	5	275	278	277	288
Interest		514	523	530	529
Total	5	789	801	807	817
Receipts less expenditures	14	−107	71	234	400

[1] Opened to traffic Dec. 13, 1947.

in toll-revenue term bonds bearing 2½ percent interest and maturing in 30 years. The issue was sold at approximately 95.75 percent of par. When later studies showed that the total cost would be greatly in excess of the original engineering estimates, an additional $5 million in term bonds bearing 2¾ percent interest was sold in 1947. These also sold at a discount. In 1949, an additional $600,000-issue of revenue bonds was sold in order to finish the construction and provide some funds to operate the facility.

The Maine Turnpike earned gross revenues of $1,217,000 during the calendar year 1951, an increase of 17 percent over 1950 and 39 percent over 1949. An indication of the adequacy of these revenues to service the outstanding $20,307,000 revenue bonds ($293,000 retired in 1951) is shown in table 14, which presents receipts and expenditures for the various years, and the net revenues available for sinking funds or bond redemption. It would appear that if the traffic and revenues of the Turnpike continue to increase it should be able to liquidate its obligations. However, it appears to have been skating on rather thin ice during the first 4 years of its operation.

In 1949 and 1950, 90 percent of the traffic on the Turnpike was passenger vehicles, which earned 84.2 percent of toll revenues in 1949 and 82.1 percent in 1950. Passenger, especially tourist, traffic is vital to the revenues of the Turnpike, and any diminution of passenger revenues that might result in the future is likely to have disastrous effects on the Turnpike's solvency.

The New Hampshire Turnpike

The New Hampshire Turnpike, extending from Seabrook on the Massachusetts line to Portsmouth, where it connects directly with the Maine Turnpike, is 15 miles in length. The majority of the traffic on this facility is of out-of-State origin, and represents primarily vacation travel en route to recreational areas in northern New England and the Maritime Provinces.

The New Hampshire Turnpike.

The State used extreme care in financing this project, and avoided the high financing charges and interest costs during construction characteristic of other toll-road facilities. The facility was constructed by the State Department of Public Works and Highways and financed by the issuance of short-term notes which were purchased by private banking houses. In April 1951 the State issued general-obligation bonds for various purposes amounting to $12.6 million of which $7 million were for the purpose of refunding the toll-road notes. These bonds (due 1952–76) bear an interest rate of 1.60 percent and were sold at a slight premium, 100.107. Thus the net interest rate to the State was only 1.588 percent.

During the first full year of operation (July 1, 1950, to June 30, 1951) the gross revenue of the facility totaled $391,000, with maintenance and operation charges amounting to $39,000. The sum of $175,000 was transferred to the sinking fund, although no interest charges on the bonds fell due in this period. On the assumption that the bond issue represents the total cost of the project, $280,000 per year will be needed for principal amortization alone.

The New Jersey Turnpike

The New Jersey Turnpike, which extends 118 miles across the State from the new Delaware Memorial Bridge on the Delaware River below Wilmington, Del., to the George Washington Bridge, which connects New Jersey with New York City, was opened to traffic in January 1952.

The construction, operation, and maintenance of the Turnpike are being undertaken by the New Jersey Turnpike Authority, established by the legislature in 1948. The first and foremost problem of the Authority was to determine the most suitable means of financing the project. The Authority was particularly anxious to avoid the heavy bond discounts and interest charges during construction which are usually associated with toll-road financing. If financing were to be carried on in the usual manner, investment underwriting groups had estimated that a total of $245 million in capital funds would be necessary to finance the cost of construction, of which approximately $25 million would have been for interest payments.

Special legislation allowed the Turnpike Authority to utilize a unique alternative plan of financing. A negotiated agreement was effected with 50 insurance companies and other institutional investors whereby as much as required of the estimated $220 million was supplied to the Authority on a forward-commitment basis. The Authority drew funds from this committed amount as construction progressed and, in turn, issued bonds of an equivalent amount, the bonds bearing an interest rate of 3¼ percent. This arrangement minimized interest charges during construction and guaranteed the sale of bonds at par. A special fee of 0.5 percent of the total commitment was paid to the investors by the Authority. The New Jersey Turnpike bonds are callable in 10 years from the date of issuance at 103, with a steadily decreasing scale of premiums for bonds called after 10 years.

In September 1951 the Authority found it necessary to ask the bondholders to approve an additional $35-million bond issue to complete the Turnpike. According to the Authority, construction costs had exceeded the original estimates and additional funds were required to effect completion. The necessary two-thirds of the bondholders granted their approval and the issue was sold in October 1951. The new issue bears an interest rate of 3.20 percent and the selling price was 98.15.

At the time that the Authority asked approval of the additional $35-million bond issue it also asked for power to issue extension bonds to finance construction of direct connections of the New Jersey Turnpike with the Pennsylvania Turnpike (estimated cost, $12 million) and with the New York Thruway (estimated cost, $30 million). The Authority is also studying the feasibility of constructing a bridge across Newark Bay to connect the Port Street, Newark, interchange to the Hudson County peninsula.

The Turnpike Authority has adopted a schedule of toll rates for the Turnpike which is somewhat different from those in effect on most toll roads. The rates charged for the full 118 miles range from $1.75 for passenger cars to $5.00 for the larger trucks. However, the toll rates on the northern end of the Turnpike are considerably higher than the rates charged on the southern section. Truck rates are rather low in comparison with those in effect on the Pennsylvania Turnpike. The low truck rates were considered necessary to attract truck traffic, as the New Jersey Turnpike offers trucks less advantages over competing routes than does the Pennsylvania Turnpike.

The New Jersey Turnpike: Newark-Jersey City interchange.

The New York Thruway

The constitution of the State of New York prohibits the State from incurring debt except to repel invasion, to suppress insurrection, to defend the State, and to suppress forest fires. Tax-anticipation notes can be issued. The constitution can, however, be amended in order to provide for incurring additional debt. Two successive legislatures must approve the amendment and it is then submitted to the electorate for final approval.

In 1942, legislation was passed which enabled the State Public Works Department to proceed with the construction of the New York Thruway extending from New York City to Buffalo. In 1945 a committee of engineers investigated the possibilities of installing a conventional toll system on the Thruway and reported that such a system would not produce sufficient funds to finance its construction, because of the impracticability of collecting tolls for the large volumes of local traffic in the vicinity of the various cities along the route, and the fact that the volume of long-distance travel alone would not produce sufficient revenue to amortize the cost of the project.

In a report of March 4, 1950, the Governor's Thruway Committee, appointed to consider a new plan for making the Thruway self-supporting, proposed the establishment of a State Thruway Authority with power to finance, construct, and operate the Thruway. The Committee recommended continuance in 1950–51 of the Thruway construction from State funds, supplemented by Federal funds wherever possible; early completion of studies, plans, estimates; and a constitutional amendment to place the State's credit back of the Authority's bonds to obtain a great savings in interest costs.

The Committee further recommended that the amortization of bonds issued to construct the Thruway should be financed by issuance of a special permit or license on an annual basis to all Thruway users. The Committee's report envisaged these stages of financing: Continue building the Thruway from State funds until the Authority's financial structure is perfected; thereafter, issue Authority short-term notes for construction; and, if the people will approve State backing of the Authority's obligation by constitutional amendment, issue Authority bonds to finance completion of the Thruway and to retire the Authority's earlier notes and pay back the State.

Thruway Authority Created

On March 21, 1950, the Governor signed the New York State Thruway Authority Act, creating a three-man board to be known as the New York State Thruway Authority. One member is designated as chairman and is the chief executive officer of the Authority. The members are appointed by the Governor with the advice and consent of the Senate. The Authority is a public corporation, and was granted all the powers necessary to construct the Thruway. It has the right to possess and use State property, but upon assuming such jurisdiction it shall be indebted to the State in an amount equal to the cost of construction by the State of Thruway improvements on such property.

The Authority has the power to issue bonds not to exceed $500 million. The term of such bonds cannot exceed 40 years in length, and the bonds cannot be sold at less than 98 percent of their face value, nor can the interest be more than 3½ percent. The Authority can issue bond-anticipation notes and has the power to renew them from time to time. All bonds issued by the Authority are to be general obligations of the Authority payable from any of its revenues. To the extent authorized by the constitution at the time bonds and notes are issued, the payment of the bonds or notes is fully and unconditionally guaranteed by the State.

The 1950 and 1951 legislatures acted favorably on a constitutional amendment authorizing the State to issue general-obligation bonds of $500 million for the construction of the Thruway, and in November 1951 the electorate gave the amendment final approval by voting overwhelmingly for its adoption. Until such time as the bonds are sold the construction expenditures are being met by proceeds of short-term notes and regular State appropriations.

On September 28, 1950, the Authority embarked on its short-term finance program. It borrowed $10 million in short-term loans at 1½ percent interest from 14 banks, backed by an agreement with two life-insurance companies to buy an equal amount of refunding long-term bonds 2 years hence. This agreement provided that if the Authority issued straight revenue bonds, forcing the banks to buy them to protect this loan, the two insurance companies would each purchase $5 million of the bonds from the banks on a 2⅞ percent basis.

At the same time the Niagara Frontier Authority, which operated two toll bridges on the Thruway route, turned their operation over to the Thruway Authority which agreed to retire the outstanding bonds of the two bridges and begin immediate construction of the Thruway in their vicinity.

On March 31, 1952, the Thruway Authority negotiated its first large loan—$60 million in short-term notes bearing an interest rate of 1.1 percent and due in September 1953 were sold to a syndicate of 21 banks. Of the proceeds, $10 million will be used to repay the loan previously discussed and the remaining $50 million will be used to advance construction of the Thruway. According to latest reports the State has already advanced $106 million to the Authority, of which $26 million are to be repaid during the 1952–53 fiscal years.

Although the first proposal for collection of fees on the Thruway was by the sale of special permits or annual licenses, this has been abandoned in favor of a combined permit and toll financing system. This latest proposal by the Authority's consulting engineers recommends toll booths at 54 of the 94 interchange points plus five toll barriers at the ends of the main toll-paying section and six barriers on the spur sections. The proposed rates are as follows (25) : Passenger cars registered in New York State, $10 annual fee or 1 cent a mile; out-of-State passenger cars, 1 cent a mile; busses, 3.5 cents per mile; trucks less than 7,000 pounds unloaded weight, 1 cent a mile; and trucks over 7,000 pounds unloaded weight, 2 to 6 cents per mile (average 3.3 cents). There would be special toll fees on the Tarrytown–Nyack Bridge across the Hudson River.

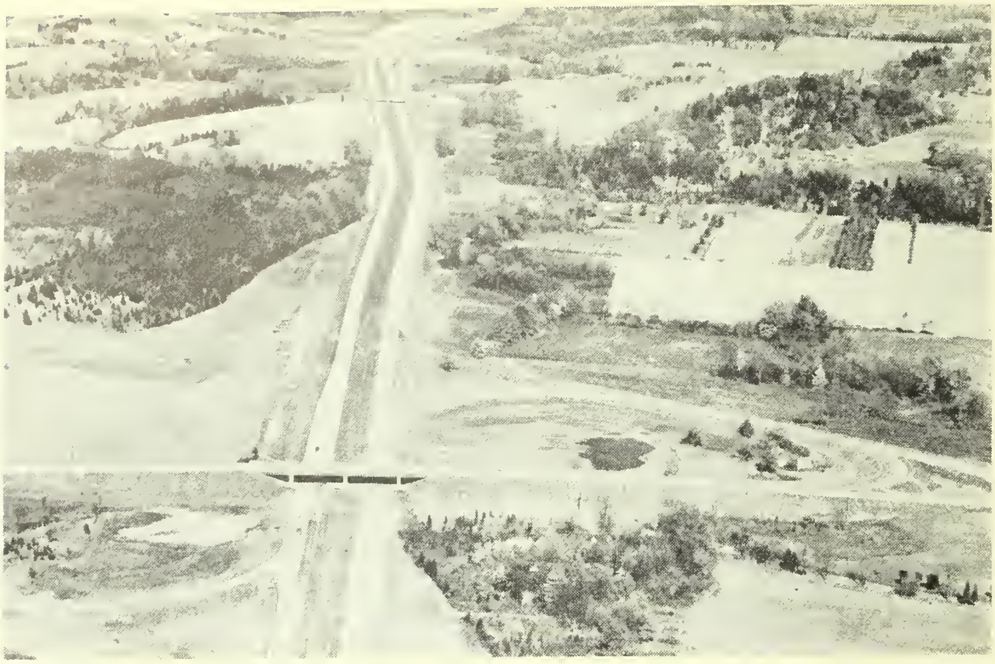
This new proposal is the result of the State's new truck weight-distance tax. Thruway officials had planned to charge truckers a flat annual fee but felt they had to change their plans to conform to the tax the trucks would be paying on other State highways.

According to the latest information available (March 1952), 41 miles of Thruway were open to traffic and 100 miles were under construction. The Thruway Authority's objective is the completion of the entire 535-mile route by 1954.

Fernandina Port Authority

Article IX, section 6, of the constitution of Florida provides that the legislature shall have power to provide for issuing State bonds only for the purpose of repelling invasion or suppressing insurrection, or for the purpose of redeeming or refunding bonds already issued, at a lower rate of interest.

Although the State of Florida has no recognized bonded debt, the State indirectly uses borrowing as a means of financing highway improvements by resorting to lease-purchase agreements with the counties, municipalities, special districts, and the Florida State Improvement Commission. As required by Chapter 21853, Acts of 1943, the Constitutional State Board of Administration administers the bonds issued for the construction of bridges or highways leased for a term of more than 1 year or purchased under installment-purchase agreements by the State Road Department. The State Road Department rentals are paid from the 80-percent surplus gasoline-tax funds[3] of the county or counties in which the road or bridge is located.


The New York Thruway near Saugerties.

On October 21, 1948, the State Road Department entered a lease-purchase agreement with the Fernandina Port Authority, effective upon completion of a toll road to be constructed by the Authority. The Port Authority issued $4.6 million in revenue bonds which bore a 4-percent interest rate. Proceeds from the sale of the bonds were to be used for the construction of a 16-mile toll road, connecting bridges, and ferry connection in Duval and Nassau Counties. Construction started March 16, 1949. The revenue bonds are not general obligations of the State or the counties and will be serviced from tolls and State Road Department rentals.

The lease-purchase agreement between the State and the Port Authority, under which the State agrees to maintain a portion of the toll road, does not become effective until the facilities have been completed and put in operation. The project was not certified as complete until February 1952, although it was opened to traffic in the fall of 1950. As the project has been operating at a deficit, the Authority defaulted on the $92,000 interest payment due in November 1951 and the State Road Department agreed to make the payment. Reorganization plans were put into effect by which the State Board of Administration took over the management of the project, along with the assets and liabilities of the Authority.

Other Toll Roads

Four other active toll-road projects are the Ohio Turnpike, the Oklahoma Turnpike, the Denver-Boulder toll road in Colorado, and the West Virginia Turnpike.

The Ohio Turnpike is by far the largest undertaking of the four. It contemplates the construction of a 241-mile road from the western terminus of the Pennsylvania Turnpike, passing near Youngstown and Akron, to Toledo, and thence to the Indiana State line. It will also pass within easy access of Cleveland. The estimated cost is $326 million. After the engineering report and the preliminary studies of the consulting engineers had been accepted by the Turnpike Commission the information was examined by the financial advisor to a group of interested investment bankers. The financial advisor approved of the project from a financial viewpoint, but advised the investment syndicate not to finance it because of a legal flaw concerning acquisition of right-of-way in the provisions of the Ohio Turnpike Act.

It appeared that the Commission could not acquire right-of-way until cash payment had actually been made for the property involved. It was feared that litigation could delay construction progress considerably. A special session of the Ohio Legislature which convened December 10 amended the Turnpike Act to overcome this objection. The amendment authorizes the Commission to take possession promptly of land condemned for right-of-way purposes. A friendly suit, filed in the Ohio Supreme Court to test constitutionality of the turnpike act and the recent amendment, resulted in a favorable decision.

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[3] The State Board of Administration receives the entire proceeds from 2 cents of the gasoline tax for the service of county road debt incurred prior to 1931 and, after providing for current debt service and sinking fund requirements, divides any surplus of these funds accruing to the credit of a county as follows: 80 percent to the State Road Department to be expended in said county and 20 percent to the county for use on the county road system. The funds so distributed to the State are commonly referred to as 80-percent surplus gasoline-tax funds.

On June 4, 1952, the Turnpike Commission sold $326 million of $3\frac{1}{4}$-percent term bonds, maturing in 40 years. The issue was sold at 97.60 and re-offered by the underwriters at par.

The Oklahoma Turnpike will include 88 miles of road between Oklahoma City and Tulsa, originally estimated to cost $31 million. Definite construction plans were completed in 1950, and $31 million of revenue bonds were sold through an investment underwriting firm at an average interest cost of 3.4 percent. Completion date of this project is set for January 1953. Increased costs made it necessary for the Authority to sell an additional $7 million of bonds in June 1952, at an average interest cost of 3.84 percent. These bonds will mature in 1990.

The Denver-Boulder toll road in Colorado, now completed, was financed from the proceeds of a $6.3-million toll-revenue bond issue sold at par, with interest ranging from $2\frac{7}{8}$ to 3 percent. The State is guaranteeing up to 30 percent of the debt service on the bonds. Construction and operation is being undertaken directly by the State Highway Department.

The West Virginia Turnpike Commission in April 1952 offered a $96-million revenue-bond issue which was purchased by a large banking syndicate. The 3.75-percent bonds were re-offered to private investors at a price of 99. Unverified reports indicate that the syndicate purchased the bonds at 95 which would mean that the bonds were discounted $4.8 million, and the net interest cost to the Commission would be about 3.95 percent. The proceeds of the issue will be used to finance the construction of that portion of the Turnpike between Charleston and Princeton, a distance of about 88 miles. The Turnpike will be the first of the modern toll roads which has only two lanes.

In other States—including Florida, Georgia, Illinois, Indiana, Kentucky, Maryland, Massachusetts, North Carolina, and Virginia—enabling legislation to establish toll facilities has been enacted and active surveys have begun on definite routes in Florida, Georgia, Massachusetts, North Carolina, and Virginia.

SPECIAL STATE AUTHORITIES

Officials in many States have been hampered in their attempts to make capital improvements by constitutional restrictions on the issuance of bonds. In most States these restrictions have not seriously impeded the credit financing of highways but the amending processes are time-consuming and the results subject to varying political winds. In some States special authorities have been created and granted corporate powers to incur debt for various purposes. These authorities issue bonds, construct facilities, and enter into leases or rental agreements with various State agencies. The rental or lease payments are set at a level that will enable the authority to make debt-service payments and handle administrative costs. The bonds of these authorities are not backed by the full faith and credit of the State.

Florida, whose constitution flatly forbids the incurrence of debt, and Pennsylvania, whose constitutional restrictions are rather rigid, are the two States in which special authorities are being used with considerable success.

Pennsylvania Authority

Act No. 128 of the General Assembly, approved April 18, 1949, Public Law 604, created the State Highway and Bridge Authority of Pennsylvania. The Authority is a public corporation and governmental instrumentality, created for the purpose of constructing and operating State highway facilities. Along with the usual rights and powers of a corporation, the Authority has been granted the following powers:

1. To acquire under agreement with the Department of Highways, with the approval of the Governor, State highways, bridges, easements, or rights-of-way necessary or desirable for carrying out its purposes, or to lease from the Department any property now owned or hereafter acquired by the Commonwealth, for a term not exceeding 99 years, at a nominal rental or at such annual rental as may be determined. The Authority can lease to the Commonwealth or to the Department of Highways any project at any time constructed by the Authority, or any property at any time acquired, or it can sell, transfer, or convey to the Commonwealth any constructed project or any property acquired by it.

2. To fix, alter, charge, and collect rates, rentals, and other charges for the use of the facilities of, or for services rendered by, the Authority, or projects thereof, for the purpose of providing the funds necessary to fulfill the purpose of the Authority, including the payment of principal and interest on its obligations.

3. To borrow money, make and issue notes or bonds not to exceed $40 million (the 1951 legislature increased this to $80 million).

4. To borrow money or accept grants or to enter into contracts, leases, and other agreements with any Federal agency.

No project can be undertaken without the approval of the Department of Highways. The Authority cannot pledge the credit or taxing power of the Commonwealth, nor are its debts to be deemed the obligations of the Commonwealth, nor is the Commonwealth liable for interest and principal payments on its obligations. No project can be leased to the Department of Highways for a period in excess of 30 years.

The Governor of the State, State Treasurer, Auditor General, Secretary of Internal Affairs, Secretary of Highways, the Speaker of the House, President pro tem of the Senate, the minority leaders in both houses, and three citizens of Pennsylvania are members of the governing board. The Authority was formally organized May 3, 1949.

In December 1949 the Authority sold $15 million in bonds at a price of 99.567. The bonds bore interest rates varying from 1.0 percent to 1.25 percent, and the basis, or effective interest rate, was 1.2534 percent. In April 1951 $25 million in bonds were sold at a price of 98.9043 with an effective interest rate of 1.5909 percent.

The first highway and bridge construction program of major projects approved by the Authority Board consisted of 22 projects at an estimated cost of $53,598,710, including $14,375,000 in Federal aid. The program was revised by action of the Authority Board on September 25, 1950, to 27 projects at an estimated cost of approximately $54 million—$39,382,117 to be provided by the Authority and approximately $15 million to be received in Federal aid. Through November 30, 1951, the Authority had made payments to contractors amounting to about $20 million.

Pottsville bypass in Pennsylvania.

Construction under way on the Jacksonville Expressway in Florida.

All projects now on the Authority's program will require a little more than $96.5 million in State Highway and Bridge Authority funds, and in excess of $31 million in Federal-aid funds. Of the total program, projects involving an estimated $43 million of Authority funds are still in the planning stage, and no Federal funds have been programed. Recent legislation has increased the Authority's debt limit to $80 million.

The State Department of Highways has met right-of-way costs on all of the Authority's projects, and the Authority has also made use of the Department's surveys, plans, and construction inspectors. Among the projects completed, under construction, or planned by the Authority, with their costs, are the North Bridge at Harrisburg, $7,063,000; the Schuylkill Expressway in Montgomery County, $19,072,000; the Penn-Lincoln Parkway, $26,914,000; and the Pottsville bypass, $2,776,000. Included in the cost figures of the Schuylkill and Penn-Lincoln projects are Federal participation funds of $6,940,000 and $13,133,000, respectively.

After each project is opened to traffic the Authority charges the motor license fund a yearly rental which will amortize the cost of the project over a 10-year period. The Authority's projects are toll free.

Florida Improvement Commission

The Florida State Improvement Commission, created by chapter 420, Florida Statutes, 1941, is composed of the Governor, the Chairman of the State Road Commission, and three appointed members. The purpose of the legislation creating the Improvement Commission, as amended, is to create a State agency primarily to make possible and facilitate the acquisition, construction, maintenance, and operation of public buildings, facilities, and works for State purposes; to assist in effectuating postwar planning and construction; to relieve unemployment; to accept and use in carrying out these purposes loans or grants of money appropriated by the Congress of the United States; and to promote the general welfare. Although the Florida State Improvement Commission is authorized to issue revenue bonds for the construction of facilities, such as airports and public buildings, of the type that can be owned and operated by an agency of the State, only the operations pertaining to highway facilities are discussed here.

Upon application by any county and approval by the State Road Department, the Florida State Improvement Commission is authorized to construct roads or bridges connecting State highways, and to finance such projects by the issuance of revenue bonds payable from revenues accruing to the Commission from the State Road Department under lease-purchase agreements. The issuance of the bonds must be approved by the State Board of Administration, which administers all bonds issued to finance the construction or purchase of bridges or highways leased for a term of more than 1 year, or purchased under installment-purchase agreements by the State Road Department from any public body, county, or other public bridge authority.

The Florida State Improvement Commission issued a little more than $9 million worth of bonds in 1947 and 1948 for roads in 14 counties. In general, the projects constructed with these bond issues were not on principal State routes and, with the exception of a $3.6-million Palm Beach County issue, the issues in each county were for $600,000 or less. The State Road Department rentals are paid from the 80-percent surplus gas tax [4] accruing to the credit of the counties in which the roads are located. The roads are toll free and are maintained by the State.

On September 25, 1950, the Improvement Commission sold a $28-million bond issue carrying a net interest rate of 2.6187 percent to finance part of the cost of construction of the Jacksonville Expressway System, consisting of interstate expressways, arterial connections, and two new bridges (the Arlington Bridge and the Interstate Bridge) over the St. Johns River. At that time, the estimated cost of the Jacksonville Expressway System was $41,818,000, of which the State Road Department was to provide $13,818,000, including Federal aid. Debt-service payments will be made from rentals received from the State Road Department under the lease-purchase agreement. The State Road Department rentals will be paid from tolls collected on the two bridges, and from the 80-percent surplus gasoline tax revenue accruing to the State Road Department for use in Duval County. Maintenance and toll-collection costs will be paid by the State Road Department from other funds. However, the State Road Department is entitled to reimbursement from the Arlington and Interstate Bridge tolls for the cost of operation, extraordinary repairs, and replacements of capital assets of these two bridges, but only after all bonds of series T (term bonds maturing in 1980) have been paid, and only after all required payments for debt service and reserves for the serial bonds have been fully made. The State Road Department is not entitled to reimbursement for such costs incurred on other parts of the system.

With the consent of the City of Jacksonville and Duval County, the system has been designated by the State Road Department as a controlled-access highway.

Definite plans have been made by the State Road Department, the Florida State Improvement Commission, and Broward County for the construction of tunnels and bridges in Broward County at a total cost of approximately $15 million of which $13 million will be borrowed and $2 million will be contributed by the State Road Department. The bonds would carry a pledge of tolls from the bridges and tunnels and the 80-percent surplus gasoline tax.

By the end of 1950 the Commission had issued over $37 million in bonds of which $36.8 million were outstanding. Inclusion of the Tampa Bay Bridge toll-revenue bonds previously discussed pushes this amount close to $60 million.

COOPERATIVE PROJECTS

In recent years there has been an increasing number of projects in which two or more governmental units have cooperated. The inclusion of specified amounts of secondary and urban funds in the Federal-aid highway acts has probably influenced this cooperativeness in that it was necessary that the State confer with the local governmental units in programing these funds. These cooperative projects vary in magnitude, and the following discussion is concerned with the financing of a few of the larger and more prominent ones.

Atlanta Expressway System

As envisioned by the plans and recommendations of consultants employed by the Georgia State Highway Department, the Atlanta Expressway System would bisect the city as near the center as possible. The two legs of the system, one running north-south and the other east-west, would relieve traffic flow into and through the city. The proposed plan, submitted in 1945, was estimated to cost over $60 million. As construction costs and real-estate values have increased considerably since that time, the total cost today is estimated at nearly $80 million.

In order to finance their shares of the program, Fulton County issued $12.5 million, and the city of Atlanta issued approximately $4 million, of traffic-improvement bonds. Urban Federal-aid and State funds of approximately $12 million have been added to the bond funds, making a total of $28.5 million available for the project. The city is now considering further credit financing proposals for the continuation of the project.

At the time the bond issues were voted it was agreed by the city and county that they would each appoint five members to a bond commission for purposes of advising the two units of government in the expenditure of the fund. This commission, known as the Joint Bond Commission, recommends the allocation of all bond funds to projects considered most essential, and their recommendations are carried out by the engineer-secretary of the commission.

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[4] For an explanation of the 80-percent surplus gas tax see footnote 3, p. 80.

The first contract was let in the latter part of 1948 and work began in early 1949. Since that time five other contracts have been let, extending the expressway portion under contract to about 12 miles. Three miles of the expressway (north-south leg) are now open to traffic, and 3½ miles more will soon be opened.

The Atlanta Expressway System is actually a system of freeways, being completely controlled-access roads with no grade crossings to interfere with free traffic flow. The downtown section is six lanes, divided, while outlying sections are four lanes, divided.

In developing this system, the city and county acquire rights-of-way and State and Federal funds are used for construction. In the downtown section it was found that the cost of right-of-way and cost of construction are approximately the same, running in general from $1.5 million to $2.0 million per mile for each.

Cook County and Chicago

For many years the traffic problem in and around the Chicago metropolitan area has been a subject of much study. Some progress had been made towards alleviating this congestion by the construction of the Outer Drive and Wacker Drive in Chicago, but these two projects aided relatively little in relieving the total problem.

By 1944 plans for a proposed system of expressways for the Chicago area were adopted and a committee of engineers representing all interested parties recommended that the projects be built cooperatively by the Federal Government, State, county, and city. The agreed-upon system comprises approximately 190 miles.

In 1946 Cook County and the city of Chicago submitted superhighway bond issue proposals to the electorate and both issues, the city's $42 million and the county's $70 million, were approved. The Illinois Supreme Court declared the county issue invalid, but the 1947 Illinois General Assembly passed enabling legislation which authorized Cook County to issue the $70 million in superhighway bonds without referendum, and further authorized the county not only to levy taxes for debt service on the bonds but also to use the county's share of the motor-fuel tax fund for debt service. To date Cook County has issued $35 million of superhighway bonds and the city has issued $29 million. The county's bonds bear an interest rate of 2½ percent, and those of the city 1½ percent.

The Atlanta Expressway, Georgia.

This cooperative program, of which the portions within the city of Chicago total 67 miles with an estimated cost of $446 million, is progressing at a fairly rapid rate. It is the unusual cooperativeness of the four participating units of government involved that is unique. Some of the projects are financed jointly by the Federal, State, county, and city governments; others by the Federal Government, State, and county; still others are joint ventures of the State, county, and city, or of the State and county. The Cook County Highway Department prepares plans and sells them to the State. The city and county buy right-of-way for the State and let contracts in which they advance the State's portion of the costs.

Michigan Revenue Bonds

Articles III and X of the Michigan constitution prohibit the State from incurring debt except to cover casual deficits ($250,000 limit), to repel invasion, to suppress insurrection, and to defend the State. By means of constitutional amendments debt in excess of the constitutional limitations can be incurred. However, the pledge of specific revenues by the State Highway Commissioner for the retirement of the limited-access highway revenue bonds provided for in Act No. 22, Public Acts of 1950, is not held to be an infringement of constitutional limitations.

This act, approved June 7, 1950, amended Act No. 205, Public Acts of 1941, providing for the establishment and maintenance of limited-access highways by inserting the word "construction" in the title and making specific provisions for financing the improvements made under authorization of the 1941 act.

These acts grant the State Highway Commissioner, boards of county road commissioners, and cities and villages, either acting alone or in cooperation with each other, or with any Federal, State, or local agency, the authority to undertake the construction and maintenance of controlled-access highways, and all other powers necessary for such accomplishments. Plans and specifications are reviewed by the State Highway Commissioner and cost estimates are obtained. After their approval, the State Highway Commissioner may enter into contract with the participating governmental units providing for the acquisition, construction, or improvement of the proposed controlled-access highways. The contracts must provide for the allocation of the share of the cost which is to be paid by each participating governmental unit, and the payment of the costs in annual installments which cannot exceed a period of 30 years. The contracts are to be executed by the State Highway Commissioner after approval by the State Administrative Board and by all other participating governmental units.

The State Highway Commissioner is authorized to make annual contributions to the cost of construction from all or any highway-user imposts and to make an irrevocable pledge of such funds, but such contributions from funds of the State Highway Department are not to exceed $3.5 million annually. The counties, cities, and villages are also authorized to make an irrevocable pledge of highway-user imposts received from the State, and of any contributions of funds received from the Federal Government, or from any other source, for the projects.

The act authorizes the governmental units that are parties to such contracts to borrow

The Edsel Ford Expressway, Detroit, Mich.

money and issue negotiable revenue bonds or notes. The bonds and notes issued under this act are secured by the irrevocable pledge of the annual contributions required to fulfill the contract agreements. The bonds are not to be general obligations of the issuing governmental units, but are to be payable from the proceeds of highway-user funds received by each of the units from the State and from other pledged funds, including Federal funds. The total amount of bonds and notes cannot, at any time, exceed $200 million. The bonds issued under this act are not construed to be a pledge of the full faith and credit of the State of Michigan. The title of the bonds will be Limited-Access Highway Revenue Bonds.

In November 1951, $80 million in State of Michigan Limited-Access Highway Revenue Bonds, Series I, were issued. The bonds, maturing from 1955 through 1976, sold at a slight premium ($40,000), and the interest cost to the State was 2.1246 percent. The State pledged a minimum annual payment of $2.5 million, and Wayne County and the city of Detroit each pledged minimum annual payments of $1.25 million for debt service. The proceeds of the issue will be used for the construction of the Edsel B. Ford and the John C. Lodge Expressways. The total cost for the completion of the two expressways is estimated at $134 million, and it is anticipated that Federal funds will make up the major portion of the remaining $54 million. The official statement, issued to prospective bond buyers prior to the sale on November 20, 1951, in outlining the purpose and need of the issue, states:

> The pay-as-you-go basis upon which the construction has proceeded thus far would, if continued, require a period of at least 15 years to complete the two expressways. All of the participating governmental units are agreed that this plan is much too slow, as a tremendous need now exists for the expressways. To expedite the construction so that they can be completed in an estimated period of not more than 5 years, the present plan of financing has been worked out.

Texas Expressways

Houston, Dallas, Fort Worth, San Antonio, and Austin, Texas, have embarked on large urban expressway projects. These projects are cooperative ventures of the cities, counties, State, and Federal Government.

The Houston Urban Expressway System, as agreed upon by the State Highway Commission and the city of Houston, will cost an estimated $60 to $75 million for construction, plus $25 million for right-of-way. The cost of the Central Expressway in Dallas is estimated at $23 million including right-of-way. The cost of the San Antonio Expressway is estimated at $10 million. No figures are available on the estimated cost of Fort Worth's 17.6 miles of expressways or for the Austin program. In all of these projects the cities are acquiring the rights-of-way and paying all right-of-way costs. In most of the projects, design and construction costs are shared by the State and Federal Governments. A downtown section of the Dallas Expressway will be constructed by the city with the State paying half the costs.

Although all these cities have incurred large amounts of street debt in recent years, none of the issues have specifically earmarked any of the proceeds for expressways, with one exception: Austin issued approximately $1 million in right-of-way bonds for expressways. Reports received from the State Highway Department, however, indicate that sizable portions of the proceeds of urban highway bond sales were devoted to right-of-way and construction expenditures on urban extensions of the State highway system. The reported facts are given in table 15. It is reasonable to assume that the bulk of the expenditures on urban extensions of the State highway system were made on the expressway systems.

Table 15.—Application of proceeds from urban highway borrowings, exclusive of refunding issues, in Texas

Year	For urban extensions of State highways			For local city streets			Total[1]
	Right-of-way	Construction	Total	Capital outlays	Equipment and other	Total	
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
1945	1,491		1,491	2,447	20	2,467	3,958
1946	3,548	[2] 1,018	4,566	11,752	35	11,787	16,353
1947	2,956	100	3,056	9,578	207	9,785	12,841
1948	2,253	42	2,295	12,827	88	12,915	15,210
1949	5,945	1,643	7,588	11,286	942	12,228	19,816
Total	16,193	2,803	18,996	47,890	1,292	49,182	68,178

[1] The net proceeds of both long-term and short-term borrowings are reported in this table while table 7 shows only the par value of long-term borrowings.

[2] Includes $695,000 for the purchase of the International Highway Toll Bridge at Laredo.

REGULAR CREDIT FINANCING OF STATE HIGHWAYS

The spectacular achievements of recent years in the fields of urban expressways and toll roads and bridges have tended to overshadow the more prosaic but nonetheless solid accomplishments that have been made possible by the judicious use of general- or limited-obligation bonds issued by certain States for the purpose of accelerating a carefully planned highway program. In these States the public evidently felt that the advantages of accelerating the highway program by issuing bonds more than offset the interest costs incident to credit financing. It was also evident that, although the highway-user tax structure had been revised in order to finance the long-range program, some of the highways were so deficient that their reconstruction was a matter of immediate concern to all interested parties and could not be delayed until such time as revenues from increased taxes became available.

Among the States that have recently issued general-obligation or limited-obligation bonds in the more or less traditional fashion are Arkansas, Delaware, Louisiana, Maine, Maryland, Massachusetts, Mississippi, Montana, New Hampshire, New Jersey, New Mexico, New York, North Carolina, Oregon, South Carolina, Vermont, Washington, and West Virginia. Examples of credit-financing activities of these types in eight States will be considered.

Maine and New Hampshire

The Maine highway-needs study, published in 1949, outlined an accelerated program requiring $27 million in bond issues. This amount was approved by referendum vote in September 1951. Bonds will be issued at an annual rate of $4 million for 6 years and $3 million in the seventh year. The entire

Table 16.—Recent highway bond issues in the Commonwealth of Massachusetts

Date of issue	Amount	Interest rate	Price	Maturities
November 1950	$60,000,000	1.25	100.00539	1951–60
May 1951	20,000,000	1.50	100.0685	1952–61
November 1951	20,000,000	1.75	100.12	1952–61
November 1951	4,000,000	1.75	100.2	1961–66

construction program, including bond proceeds and Federal aid, will average $14 million annually for 7 years and $8.4 million for the following 12 years. Deficiencies on 1,367 miles of the Maine State highway system will be corrected, together with those on 233 miles of Federal-aid secondary roads not on the State system. The first $4-million bond issue will probably be sold during the fiscal year 1953.

The New Hampshire Department of Public Works and Highways has outlined a 15-year program for correcting deficiencies on 3,700 miles of State primary and secondary roads at a cost of $90 million. In 1951 the gasoline tax was increased from 4 cents to 5 cents per gallon in order to pay interest and principal on a $14-million bond issue. It is expected that the legislature will authorize further bond issues as the program progresses. Since 1947 New Hampshire has been financing a large part of its construction program by issuing short-term notes which may be funded into long-term bonds at a later date. In 1947, 1949, and 1951 note authorizations of $7 million were made for State highways. These short-term notes are in addition to the $14-million bond issue authorized in 1951. Altogether, $26 million of highway and turnpike bonds have been provided for.

Massachusetts Financing

The Commonwealth of Massachusetts, towards the end of World War II, was confronted with the problem, common to many States, of coping with the anticipated tremendous increases in traffic with a highway system inadequate for even wartime traffic. Officials of the Department of Public Works were fully aware of the staggering highway needs; but because of the unpredictability of future appropriations they could not plan a cohesive long-range highway program. Diversion of highway funds to nonhighway purposes was another factor contributing to this lack of financial ability, as was also the relatively low motor-fuel tax of 3 cents per gallon.

In view of this situation, long-range highway-needs studies were undertaken and reported to the legislature in 1948. There were two reports, one dealing with the Boston metropolitan area and one with the State highway needs exclusive of the Boston area. The plans recommended for adoption by the two reports carried a price tag of $662 million, almost equally divided between the Boston metropolitan area and the remainder of the State. The report *Master Highway Plan for the Boston Metropolitan Area* prepared by Charles A. Maguire and Associates, consulting engineers, recommended the completion of the program within 10 years and that current revenues be supplemented by the issuance of bonds.

Chapter 306, Acts of 1949, approved in May 1949, authorized a highway bond issue not to exceed $100 million. The act specified the areas in which most of the proceeds of the bonds were to be used: $8 million to be expended by the Metropolitan District Commission; $37 million for projects in the Boston metropolitan area; $53 million for projects exclusive of the Boston metropolitan area; and $2 million for traffic safety devices on State highways and on roads constructed under section 34, Chapter 90 of the General Laws.

In July 1950 chapter 685, Acts of 1950, authorized another $100-million highway bond issue. Again, the legislature specified the areas in which the proceeds of the issue were to be expended—the same as for the preceding authorization except that the third item was increased to $54 million and the fourth reduced to $1 million.

In November 1950 the State issued $60 million of highway-improvement bonds. In both May and November 1951, $20 million in bonds were issued, completing the issue authorized by the 1949 act. Also, in November 1951, $4 million of the 1950 authorization were issued. Table 16 gives the detail on the highway bonds issued to date.

Debt-service payments are made from the highway fund, but the bonds are general obligations of the State. The 1951 legislature increased the motor-fuel tax from 3 cents to 4.3 cents in order to provide the necessary debt-service funds.

The careful planning preceding the issuance of the bonds enabled the Department of Public Works and the Metropolitan District Commission to initiate the bond construction program without delay. Under the provisions of the 1949 act the Department was authorized to expend $92 million. As of December 1, 1951, construction costs on projects under contract or completed were estimated at $65,367,300 and right-of-way costs at $17,169,900—a total of $82,537,200. This does not include engineering costs or the $2 million expended on traffic safety devices. The 1950 authorization act contained essentially the same provisions as the 1949 act. Progress under this second bond authorization to December 1, 1951, shows $22,185,700 in projects completed or under contract and right-of-way expenditures of $2,229,500—a total of $24,415,200. This total also excludes engineering costs and the traffic safety devices appropriation of $1 million. The Metropolitan District Commission, as of January 1, 1951, had expended approximately $5 million of the 1949 bond funds. It can be assumed that the remaining funds have been at least obligated by this time. As of July 31, 1951,

Storrow Memorial Expressway, Boston, Mass.

over $19 million of Federal-aid funds had been allotted to this program.

Altogether, the Department of Public Works has or will have constructed approximately 125 miles of modern highways with the proceeds of the 1949 highway improvement bonds. One of the most spectacular projects is State Route 128, the circumferential route around the city of Boston, constructed at a cost of $20 million. It was opened to traffic in August 1951, and now carries an average daily traffic of 23,500 vehicles. Another is the John F. Fitzgerald Expressway in Boston, referred to in its planning stage as the Boston Central Artery, which will cost an estimated $30 million, exclusive of right-of-way.

If shortages of material do not delay the program too much it is believed that the Commonwealth will be able to correct some of the major deficiencies of its highway system within a very short period of time. The tremendous accomplishments of the bond-issue program are also being accompanied by a fairly large construction program financed with current revenues. The Massachusetts legislature recently authorized an additional $200-million highway bond issue and created a toll-road authority to construct a toll road from Boston to the western border of the State (at an estimated cost of $150 million).

Maryland Program

Maryland has used credit financing for State highways since 1908, and by 1921 had issued more than $23 million in highway bonds. By that year the entire State highway system had been surfaced with gravel or higher-type material. The design of early Maryland roads was, however, soon outmoded by changing traffic conditions. The State continued to supplement its current revenues with bond proceeds almost without interruption until 1941.

Maryland bonds until 1933 were full-faith-and-credit obligations and were supported, for the most part, by property-tax levies. The issues in 1933 were supported only by specific highway-user taxes as enumerated in the enabling legislation.

Issues of $4 million in 1933 and $3 million in 1935 were limited-obligation bonds for widening and improving already surfaced State roads. The interest record of these subsequent issues has been satisfactory and in line with interest trends for other high-grade State bonds. The 1933 and 1935 issues carried coupon rates of from 2¼ to 4 percent. These issues were refunded in 1938 at 3 percent. The 1941 issues had rates of from 1½ to 2½ percent.

The State's postwar program has relied heavily on issues of these limited-obligation bonds. An authorization of $100 million was made in 1947, from which issues of $25 million each were made in 1949, 1950, and 1951. The issues sold above par and at net interest costs of 1.5, 1.45, and 1.73 percent, respectively.

The legislation authorizing this issue stipulated that at least 50 percent of the bond funds were to be used to pay the cost of financing, planning, and constructing projects which have an average traffic of 3,000 or more vehicles per day. Under the authority of this legislation the State Roads Commission (*26*) has indicated that expressway standards will be applied to routes having not less than 5,000 vehicles per day, and that controlled access in lesser degree will be applied to improvements on routes having not less than 3,000 vehicles per day. Thus at least half of the $100-million issue is specifically dedicated for controlled-access arterial highways or expressways.

Among the many outstanding projects on which bond funds are being used are the Baltimore-Washington Expressway, the Washington-Annapolis Expressway, the improvement of U S 50 on the Eastern Shore and the improvement of U S 40 west of Baltimore.

North Carolina Bonds

North Carolina is one of four States where all, or nearly all, rural roads are under State control. Thus the State has had to spread its available road-user funds over more than 65,000 miles of State-administered roads and streets—by far the largest network of State highways in the country. The State was also one of the first to envisage and execute a State-wide highway program and borrowed $115 million from 1921 to 1928 to construct the initial main highway system.

After assuming jurisdiction of all roads in 1931, the State found that available current revenues had to be so widely spread that any major improvement accomplishments were almost impossible. In 1949 the voters approved the issuance of a $200-million bond issue, to be financed from the proceeds of an additional 1-cent gasoline tax, to make possible the hard surfacing of 12,000 miles, and the stabilizing of 35,000 miles of secondary or rural roads in the State. It was planned to complete the program within 4 years, beginning as soon as possible after June 1949. In addition to the pledge of gas-tax revenues, these bonds are further secured by the full taxing power of the State.

By October 1, 1951, a little more than 2 years after the bond issue had been authorized, more than 9,200 miles of hard surfacing had been placed on secondary roads, and over 11,600 miles had been stabilized. Not all of this construction has been done with bond proceeds, but a recent sample indicates that about 81 percent of new hard-surfaced mileage has been built with bond funds.

Because of the varied topography and geology of North Carolina, the cost and types of secondary road pavements and bases used vary widely. Some roads have been improved at a cost of less than $3,000 per mile, while on others the costs have approached $30,000 per mile.

The cost of bituminous surface treatment or 1½- to 2-inch plant-mixed bituminous concrete on an existing road ranges from less than $3,000 to more than $10,000 per mile, depending on the cost of conditioning the existing road as a base. The general average for this type of work is about $5,000 per mile.

Sand-asphalt surfacing 1½ to 2 inches thick, which is widely used in the eastern section of the State, costs from $4,000 to $10,200 per mile, depending on the amount of base work necessary. The general average for this type of work in the eastern part of North Carolina is about $7,000 per mile.

The cost of traffic-bound macadam with bituminous surface treatment, and bituminous surface treatment on soil or portland-cement stabilized base, varies from less than $6,000 per mile to nearly $30,000 per mile, depending on the amount of grading, the nature of the subgrade, and the availability of materials. The general average for this type of work, including grading, is in the neighborhood of $12,000 per mile.

These costs, which are based on contract awards, do not include the cost of bridges, rights-of-way, and special items and may not include some preparatory work done before the contracts were let.

As of October 1, 1951, the entire issue of $200 million had been sold, at an average interest rate of 1.69 percent. About $149 million of the proceeds had been allocated to definite projects and programs, and the total expenditures from the bond fund at that date were $112,287,000.

Although the bond-issue program was exclusively for secondary roads, the availability of the bond funds has so relieved the demands for normal secondary road needs as to make possible important benefits to the 11,000-mile State primary system. Road-user revenues otherwise available for the secondary system have been released for expenditure on the primary system, thus permitting a number of important and badly needed major improvements to be completed several years ahead of normal expectancy. Although increased revenues will account for a portion of the benefits, the following comparison illustrates how the availability of the bond funds has aided construction on the primary system. During the 4-year period ending January 1, 1949, only $1.7 million of surplus funds could be transferred to the primary highway system for construction purposes. Since that time $18.5 million have been made available from surplus highway funds for construction on the primary system.

West Virginia

West Virginia has followed a pattern somewhat similar to that of North Carolina in both its highway administration function and bonding program for secondary roads. The West Virginia electorate approved in 1949 a constitutional amendment authorizing issuance of $50 million of secondary-road bonds, and the State Road Commission scheduled a 4-year improvement program to utilize these funds. A distribution formula passed by the legislature allocates to each of the State's 55 counties a flat $200,000 plus a pro-rata share of 80 percent of the remainder of the bond fund, not to exceed a top limit of $1.1 million, based on the total county unimproved secondary road mileage. Under this plan the Commission will allocate about $40.6 million on the formula basis and retain slightly over $9 million to be used at its discretion.

On October 1, 1951, slightly beyond the midpoint of the West Virginia 4-year program, approximately 2,700 miles of road improvement had been completed out of a total of 3,010 miles authorized for construction. At this rate it is expected that the entire mileage will have been completed early in 1952. Authorized expenditures as of October 1, 1951, totaled $33,217,000, segregated as follows:

Surveys and plans, 1,098 miles	$1, 210, 000
State force construction, 2,120 miles	16, 275, 000
Contract construction, 718 miles	12, 284, 000
Prison labor construction, 162 miles	2, 499, 000
Right-of-way expenditures	949, 000

During 1949, 1950, and 1951 a total of $28.5 million of secondary-road bonds were sold at a net interest cost of approximately 1.4 percent. Maximum maturity on these issues was 15 years.

Debt-service requirements on the bonds are being met with the proceeds of the 5-cent gasoline tax but the bonds are also full faith and credit obligations of the State.

Like North Carolina, West Virginia is responsible for the maintenance and improvement of all former county roads, thus giving the State a combined primary and secondary system of 32,000 miles, the fifth largest in the country. To the extent that funds for secondary roads are currently being provided from bond proceeds the corresponding highway-user tax revenues are being released for expenditure on the 4,500-mile primary system.

Washington

Until recently the State of Washington had, by tradition, always been committed to a pay-as-you-go policy insofar as financing the construction of the State highway system was concerned. Over 30 years ago the electorate turned down a proposed $30-million highway bond proposal. Except for a portion (21.8 percent) of the Emergency Relief Bonds of 1933 which was charged to State highway construction, the State had never incurred any debt for highway purposes. In 1951, however, the State deemed it necessary and expedient to break with this traditional policy and enter the field of credit financing of highway construction. The abandonment of the current-revenue policy was one of the results stemming from the reports, *Highways in Washington's Future* and *Financing Washington's Highways, Roads, and Streets* (27, 28). These reports were used by the Joint Fact-Finding Committee on Highways, Streets, and Bridges of the State of Washington in making recommendations to the 1949 legislature. Most of the committee's recommendations were adopted by the legislature and resulted in revising the motor-vehicle fee schedule and raising the gasoline-tax rate, thus greatly increasing the revenues from highway-user imposts.

It was soon apparent that relief of the most serious deficiencies would not be possible with normal motor-vehicle revenues, especially those on U S 99 and the Snoqualmie Pass on U S 10. Although other projects were included in the bond program the necessity for expanding capacities on these two routes motivated the 1951 legislature in its authorization of a $66,703,625 bond-issue program. The act authorizing the bond issue specifically earmarked the projects upon which the proceeds of the issue were to be expended: $33,500,000 for reconstruction of U S 99 between Everett and the Interstate Bridge at Vancouver; $15,750,000 for reconstruction of U S 99 between Everett and the British Columbia line; $6,500,000 for the bridge across the Columbia River at Pasco; $4,250,000 for widening Snoqualmie Pass to provide additional lanes of traffic; $5,000,000 for county roads to serve Columbia Basin lands in Grant, Franklin, and Adams Counties; and $1,703,625 for retiring bonds on the Agate Pass Bridge.

In August 1951 the State issued the first block of the authorized bonds. The $12-million issue of Motor Vehicle Fuel Tax Revenue Bonds, Series A, sold at a slight premium with a net interest cost to the State of 1.8995 percent. The bonds are not general obligations of the State but the debt-service payments constitute a first and prior charge against all motor-vehicle fuel-tax revenues.

The purpose of the Department of Highways is to complete the entire bond program in 4 years; this will mean that the normal dollar volume of contracts will be almost doubled in each of the 4 years. As of December 1951, contracts upon State highways (exclusive of the Columbia Basin roads) had been awarded in total amount of approximately $5 million and the procurement of rights-of-way was actively in progress. It is apparent that Washington's highway users will soon be enjoying the benefits of this accelerated program.

Oregon

The report *Highway Transportation System in Oregon* (*29*) submitted in 1948 to the Legislative Interim Committee for the Study of Highway Road and Street Needs, Revenue and Taxation, recommended a $705-million construction program for all roads and streets in the State of Oregon. The recommended program was divided into two classifications according to needs—immediate needs and 10-year needs. Of the immediate needs construction, $150 million was on the State primary system. As annual current revenues of about $15 million were not sufficient to relieve the intolerable deficiencies of the system, the 1951 legislature authorized a $40-million bond issue to accelerate the program.

The authorization makes no designation of the projects or highways on which bond funds are to be used but does limit the amount to be issued in any one year to $15 million. The bonds are general obligations of the State. The legislature further stipulated that the bonds should be 20-year serials payable in equal installments beginning the year after the bonds were issued, and that the effective average rate of interest was not to exceed 2½ percent.

As the expenditure of the funds was left to the discretion of the Highway Commission, it was decided to utilize the bond money on interstate routes and a few others of the more heavily traveled routes; also on especially large projects where the cost involved is larger than can be absorbed in the limited current-revenue programs. Of the $40-million authorization, $22.8 million have been programed and approximately $10.5 million worth of projects have been let to contract.

In September 1951 and again in February 1952 the State sold $15 million of the bonds at effective interest rates of 1.6152 and 1.63476 percent, respectively.

The State Highway Department estimates that within 3 years, with the $15 million in current revenues combined with the bond funds, it will complete half of the immediate needs program.

The State is planning to utilize Federal-aid funds for the retirement of the bonds.

CONCLUSION

In this article we have inquired briefly into the principles of public credit as applied to highways. Both logic and experience lead to the conclusion that credit financing is justified as a means of accelerating the improvement of the highway plant and making the benefits of adequate highway transportation available to the public at an earlier date. The most advantageous situation for the use of bond-issue financing is one which requires a short period of abnormally high construction activity, to be followed by a period of relatively low construction expenditures, during which replacement needs accumulate slowly and revenues are available for retirement of the bonds.

A review of credit financing of highways as it has actually been going on during the past few years discloses considerable variety in the methods used in different States, and sometimes in the same State. Toll-revenue financing of major routes in certain States now holds the spotlight. Other States are proceeding to make it clear that toll-free financing of expressways and other major arterials is not a dream but a reality. General-obligation bonds retain their popularity in a number of States, because of the virtual certainty that they will be marketed at the rates most advantageous to the issuing government. Debt limitations and other barriers to general-obligation financing have increased the popularity of limited-obligation bonds, secured only, or chiefly, by the proceeds of road-user taxes. Obligations of this type have been issued in some States as State highway bonds differing little from general-obligation issues. In other States special State authorities with corporate powers have been created for the purpose. In still others cooperative arrangements have provided for the issue of such securities by counties or cities.

A final thought is suggested by this analysis. Roads and bridges are built as a result of demand. More or less accurate measurements of demand in dollar terms provide the justification, or economic warrant, for highway improvements. The results of such evaluations, confirmed by the experience of toll facilities, indicate that motorists and commercial users are willing to make rather high premium payments for premium service such as is provided by a controlled-access facility, whether toll or toll-free. Under these circumstances there is a natural temptation to think in terms of exploiting the money-making possibilities of traffic demand, rather than to adhere to the austere principles of public service. Wise and faithful stewardship demands that the public, as well as the investor, be given a fair return on its money, whether that money is collected in taxes or in tolls. Recent efforts to improve the terms of financing toll facilities, either by general-obligation issues or by other means, suggests an increasing awareness of these principles of good government. With continued adherence to high standards of public service, time and experience will resolve the major differences of opinion regarding the best methods of credit financing for highways.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUGUST 31, 1952

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$5,917	$31,656	$16,139	457.2	$8,818	$4,440	173.0	$22,976	$11,810	401.4	$63,450	$32,389	1,031.6
Arizona	747	3,647	2,424	87.3	1,142	507	17.8	9,338	6,139	110.5	14,127	9,070	215.6
Arkansas	3,096	9,871	5,260	286.1	3,817	1,877	94.4	16,753	8,529	492.2	30,441	15,666	872.7
California	1,871	13,877	4,948	129.1	11,769	6,169	50.4	92,043	43,371	241.3	117,689	54,488	420.8
Colorado	3,517	5,260	2,922	105.6	2,112	1,169	62.4	12,563	6,399	245.1	19,935	10,490	413.1
Connecticut	4,781	3,649	1,926	13.0	1,572	785	2.2	11,617	5,891	24.0	16,838	8,602	39.2
Delaware	891	1,734	867	6.9	950	472	3.2	7,011	3,545	50.1	9,695	4,884	60.2
Florida	1,674	18,982	9,736	268.0	6,363	3,246	161.4	15,209	7,716	225.2	40,554	20,698	654.6
Georgia	2,832	17,274	8,863	410.4	8,515	4,329	115.8	36,266	17,296	581.7	62,055	30,488	1,107.9
Idaho	3,499	9,380	5,649	263.0	2,644	1,666	65.4	8,093	5,134	139.7	20,117	12,449	468.1
Illinois	8,481	43,353	23,958	376.0	17,775	9,224	139.7	82,222	41,600	734.9	143,350	74,782	1,250.6
Indiana	6,716	36,446	18,351	177.2	9,082	5,235	195.2	28,044	14,636	315.1	73,572	38,222	687.5
Iowa	2,076	12,461	6,732	301.8	4,127	2,119	205.8	22,232	11,116	1,008.1	38,820	19,967	1,515.7
Kansas	5,484	9,115	4,184	913.3	6,649	3,358	474.7	16,388	8,541	702.0	32,152	16,083	2,090.0
Kentucky	866	18,316	9,868	302.4	5,850	2,967	134.6	18,086	9,024	332.0	42,252	21,859	769.0
Louisiana	2,252	16,780	8,043	109.0	10,285	4,781	21.6	19,230	9,593	203.6	46,295	22,417	334.2
Maine	720	7,410	3,939	52.4	3,285	1,663	28.4	10,469	5,258	83.8	21,164	10,860	164.6
Maryland	4,814	9,412	4,393	69.6	1,617	820	41.6	10,766	5,756	30.8	21,795	10,969	142.0
Massachusetts	2,999	5,820	3,055	18.5	2,237	901	.4	51,575	25,265	46.5	59,632	29,221	65.4
Michigan	1,889	24,460	12,361	398.0	9,993	5,004	245.4	62,280	27,305	350.0	96,733	44,670	993.4
Minnesota	3,428	7,927	4,217	918.2	4,115	2,160	490.0	30,862	15,998	1,150.7	42,904	22,375	2,558.9
Mississippi	2,870	16,046	8,083	535.8	5,081	2,667	170.0	15,532	8,140	445.7	36,659	18,890	1,151.5
Missouri	9,073	24,855	12,654	759.6	13,979	6,989	222.6	38,588	20,210	589.1	77,422	39,853	1,571.3
Montana	6,854	9,295	5,374	288.6	5,272	3,108	57.3	17,523	10,548	333.1	32,090	19,030	679.0
Nebraska	11,007	12,100	6,356	567.8	3,960	1,949	107.4	19,627	9,735	648.9	35,687	18,040	1,324.1
Nevada	1,847	7,727	5,854	248.2	718	587	9.4	4,616	3,847	215.7	13,061	10,288	473.3
New Hampshire	1,204	4,769	2,374	26.7	1,413	889	10.4	4,590	2,295	30.3	10,772	5,558	67.4
New Jersey	1,537	9,767	4,801	40.8	8,329	4,164	6.3	30,879	15,272	33.9	48,975	24,237	81.0
New Mexico	1,356	3,911	2,503	121.3	1,563	1,003	56.8	10,953	6,968	297.1	16,427	10,474	475.2
New York	19,227	89,415	46,572	199.2	32,453	15,395	89.8	120,934	55,229	473.9	242,802	117,196	762.9
North Carolina	3,316	23,696	11,519	329.7	5,433	2,601	119.9	24,258	11,893	476.3	53,387	26,013	925.9
North Dakota	1,974	5,598	2,931	886.7	3,662	1,876	509.8	11,392	5,708	842.8	20,652	10,515	2,239.3
Ohio	8,542	26,958	12,952	108.2	17,556	8,380	55.5	81,383	41,213	199.1	125,897	62,545	362.8
Oklahoma	3,056	12,718	7,039	171.1	7,892	4,071	167.9	20,020	10,652	217.3	40,630	21,762	556.3
Oregon	772	2,529	1,356	4.5	2,600	1,563	51.5	16,417	8,946	234.9	21,546	11,865	290.9
Pennsylvania	5,917	24,562	12,274	63.7	28,030	13,987	89.8	77,086	38,289	158.2	129,678	64,550	311.7
Rhode Island	746	4,489	2,244	33.1	785	392	2.9	18,493	9,624	31.5	23,767	12,260	67.5
South Carolina	1,752	12,777	6,900	287.4	2,889	1,470	175.7	15,476	7,874	363.7	31,142	16,244	826.8
South Dakota	795	6,463	3,785	513.9	3,005	1,828	223.8	11,955	6,894	590.5	21,423	12,507	1,328.2
Tennessee	2,970	11,649	5,500	378.5	13,099	6,237	253.5	29,901	14,338	430.5	54,649	26,075	1,062.5
Texas	9,645	3,248	1,498	136.9	11,493	6,436	309.2	61,448	32,274	1,150.0	76,189	40,208	1,596.1
Utah	776	4,658	3,568	135.7	1,537	1,145	29.4	9,034	6,797	147.7	15,229	11,510	312.8
Vermont	1,025	4,102	2,235	34.5	599	307	6.7	7,547	3,782	62.3	12,248	6,324	103.5
Virginia	2,112	20,812	10,007	273.5	8,672	4,234	132.5	24,193	12,114	321.6	53,677	26,355	727.6
Washington	1,996	11,622	5,736	133.3	2,564	1,364	107.6	14,898	7,647	145.7	29,084	14,747	386.6
West Virginia	2,410	11,919	6,009	93.5	3,982	2,030	47.6	14,583	7,267	126.1	30,484	15,306	267.2
Wisconsin	4,507	12,469	7,024	220.0	10,285	4,568	207.0	35,195	17,241	510.7	57,949	28,833	937.7
Wyoming	157	2,594	1,701	61.7	1,355	908	49.0	9,569	6,301	189.7	13,518	8,910	300.4
Hawaii	1,524	4,188	1,934	16.1	922	458	2.3	12,832	5,347	29.9	17,942	7,739	48.3
District of Columbia	1,056	13,459	5,909	1.4				4,435	2,214	.5	17,894	8,123	1.9
Puerto Rico	5,079	6,330	3,000	39.2	3,123	1,523	15.7	11,315	5,334	36.2	20,768	9,857	91.1
TOTAL	183,652	711,555	367,527	12,373.6	324,968	165,021	6,010.7	1,358,695	683,915	16,801.6	2,395,218	1,216,463	35,185.9

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DECEMBER 1952

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

Electron Microscopy of Soil Clays and Related Materials

BY THE PHYSICAL RESEARCH BRANCH,
BUREAU OF PUBLIC ROADS, AND
THE NATIONAL BUREAU OF STANDARDS

Reported by EARL B. KINTER,[1]
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Physicist

The importance of soil properties in the design, construction, and maintenance of highways has made their measurement and evaluation of primary concern to the engineer. Wide recognition that much of the physical behavior of soils stems from their clay constituents has long indicated the need for a better fundamental knowledge of these complex materials. Basic research on the soil clay-mineral content offers the most logical approach to this goal. Electron microscopy provides a visual means for investigating the size, shape, and structure of the minute particles present in soil clays and is an aid in identifying the constituents. This article reports an exploratory examination made with an electron microscope of selected clay minerals, soil clays, and related materials. Objectives of the study were to make a general examination of a broad variety of soil clays to provide basic data for subsequent correlation with information obtained from other analytical methods and performance tests, and to estimate the potential usefulness of this research tool in soils engineering work.

IN RECENT YEARS, investigators of the properties of soil materials and their application to highway work have become increasingly concerned with the constituents of soil clays. It is definitely known that clay substances, even when present in relatively small quantities, exert a strong influence on soil properties. An estimate simply of the amounts of clay contained in soils is of limited application, however, since two soils having the same approximate clay content frequently possess wide variations in properties. It has been evident that the clay fractions themselves are inherently different, and that a better understanding of soils and their properties and uses should result from an increased knowledge of the nature and properties of the constituent substances of soil clays.

Although the study of clays has been going on for many years, efforts have been seriously handicapped for want of direct and adequate scientific tools. Within the past two decades, however, considerable advances have been made through the application of modern techniques. For instance, it has been established that earthen clay materials, including the clay portions of soils, consist chiefly, although not entirely, of minute crystalline particles of hydrous aluminum silicates (*13*)[3]. These have been called the clay minerals.

A number of individual members of three principal clay mineral groups or families have been identified, and many details of crystal structure and its relation to their properties are well agreed upon. In general, however, this field of study remains in a rapidly growing stage, from both the fundamental and the practical aspects.

Of the methods applied to the problems of identifying and characterizing the constituents of clays, those of X-ray diffraction, differential thermal analysis, specific surface area measurements, and chemical analysis (including base-exchange studies) have been especially important. In all these methods the investigator is faced with many difficulties arising from the heterogeneous and impure nature and the small particle size of the material under study. Each method has supplemented the others and not one of them has afforded complete characterization. A further and quite different means of attack has been made available by the advent of the electron microscope, which, through its high resolving power, has made possible the advantages of visual examination of individual soil clay particles.

The first application of electron microscopy to clay minerals was made in Germany (*10*) and, after 1940, was extended in the United States (*9*, *15*, *18 19*). Examinations of certain soils with the electron microscope have been reported (*6*, *16*, *17*, *22*, *24*), but an intensive application to a large number of soils has apparently not heretofore been made. This report describes a preliminary study in which a number of soil clays, clay minerals, and related materials were examined with an electron microscope.

The study was of an exploratory nature, with two main objectives: (1) a general examination of a broad variety of soil clays with respect to their particle size, shape, structure, and identification; and (2) an estimate of the potential usefulness of the electron microscope in soils engineering work.

Materials Examined

A selection of 95 soil clays, clay minerals, and associated materials was examined in the microscope and micrographs were obtained. The soils, most of which are referred to by accepted soil names, cover a wide range of conditions of soil formation and parent materials. In most cases, they were subsoils containing relatively little organic matter. Many of them were from stock samples of the Bureau of Public Roads, and a few were from private collections of the authors.

The minerals examined represent almost all of the individual members of the clay-mineral groups having generally accepted mineralogical names, and a small number of non-clay minerals likely to occur in soil clays. Where available, two or more examples of the individual clay minerals were included. These were obtained from Dr. Ralph E. Grim of the University of Illinois, the American Petroleum Institute, several mineral supply houses, the Harbison-Walker Refractories Company and other commercial suppliers of clays and ceramic materials, the National Museum, and the collections of the authors. In view of the difficulties involved in obtaining authentic specimens, especially of the clay minerals, the authors wish to express their sincere thanks to all those who made samples available.

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[1] Physical Research Branch, Bureau of Public Roads.
[2] National Bureau of Standards.

[3] Italic numbers in parentheses refer to references listed on page 100.

The particle size limits of the clay fraction of soils are not unanimously agreed upon. In the Bureau of Public Roads classification, 5 microns (0.005mm.) are taken as the maximum diameter of clay, and the fraction below 1 micron (0.001 mm.) is referred to as colloid material. The U. S. Department of Agriculture and the International Society of Soil Science designate clay as that fraction below 2 microns. In any case, for visual considerations, clay particles are well below the range of the unaided human eye, which cannot distinguish between resolved objects separated by a distance smaller than 200 microns (0.2 mm.). Under optimum conditions, the absolute limit of resolution obtained with the light microscope is 0.2 micron (0.0002 mm.). For practical purposes, however, its effective limit is much above this.

The Electron Microscope

One of the fundamental factors that limit the resolving power of any microscope is the wavelength of the radiant energy illuminating the object. Theory and experiment demonstrate that about one-half of the wavelength of the illuminating source sets an absolute and unalterable lower limit to the inherent resolution obtainable. In contrast to the relatively long wavelengths of visible light, the electron microscope utilizes the extremely short wavelengths associated with a beam of electrons. Although the theoretical limit of resolution has not been achieved, the electron microscope, since its inception in 1932, has been developed to the practical limit of resolving power—about 20 Angstroms ($1A = 10^{-4}$ micron $= 10^{-7}$ mm.). This represents a hundred-fold advantage in resolving power over that of the light microscope.

Figure 1 illustrates the functional components of the light microscope and the magnetic-type electron microscope. In the light microscope, lens L_1 serves to focus the illumination from its source (a lamp, in this case) on the subject S. The objective lens L_2 forms a real, inverted, enlarged image I_1. From this first image the eyepiece lens L_3 forms a virtual, inverted, greatly magnified image I_2.

In the electron microscope, as seen in figure 1, a hot cathode emits a cloud of electrons, a portion of which is accelerated through an electrostatic field and through the anode aperture to form a divergent beam of electrons. The first magnetic coil L_1 serves as a condensing lens to collimate the beam which is directed on the subject S. Rays penetrating the subject are focused by a second magnetic coil L_2 to form an enlarged real image I_1, a section of which is further enlarged by the third magnetic coil L_3 to form a final real image I_2 on a fluorescent screen or a micrograph on a photographic plate.

Although the components of the two instruments fulfill similar functions, such phenomena as polarization or color are not possible with the electron microscope. Transmitted electrons form real images that can be viewed on a luminescent screen or photographically recorded in black and white. In electron microscopy, entirely new methods for preparing specimens and different concepts for interpreting the observed electron-optical images are required. Because of the poor penetrating power of electron beams, only small and very thin objects can be examined and, further, the examination must be made with the object under high vacuum.

Preparation of Samples

Samples for this study were prepared as dilute suspensions by dispersing a small quantity of untreated material in distilled water in a Waring Blendor, after a preliminary soaking period of an hour or more. In some cases, a drop or two of dilute ammonia water was added before stirring. The suspension was separated into the desired particle-size ranges by the customary centrifugal method. Although a thorough study of particle-size distribution and frequency was not undertaken at this stage of the work, certain separations of the soil-clay fractions were made, including the following size ranges: 10 to 1 microns; less than 2 microns; less than 1 micron; and less than 0.2 micron. Separation of the 10-to-1-micron fraction was discontinued after a few preparations because most of the particles were too massive and opaque (nontransparent to the electron beam) for adequate examination (fig. 7*D*) and obviously bore little relation to the clay fraction.

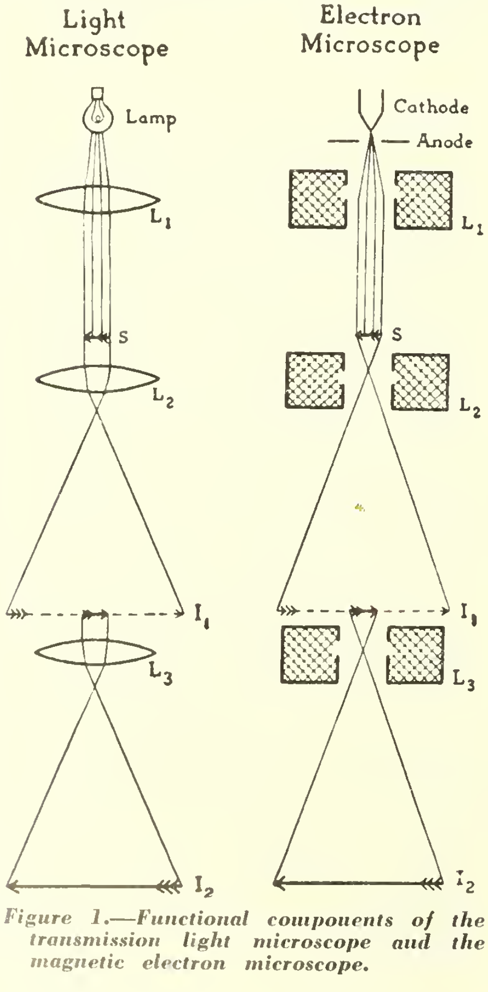

Figure 1.—Functional components of the transmission light microscope and the magnetic electron microscope.

The 1-micron separation was also rejected because its upper limit frequently proved to be too low to include all the clay-mineral content. The separations containing particles smaller than 0.2 micron were examined in some soils but usually appeared as a finer counterpart of the coarser clay. In many cases, material which was obviously not one of the hydrous clay minerals was noted to occur down through the range of this fraction. Finally, separation at 2 microns was found to be most satisfactory for the general examination desired in this work, since it included virtually all of the clay-mineral content and, near its upper limit, a representation of particles of the more rock-like minerals common to the nonclay fractions. For a more detailed investigation of the finer clay particles, separations other than those employed here would be both desirable and necessary.

After dispersion and centrifugation, the samples were further diluted with distilled water to make a faintly cloudy suspension which was stored in a small, capped vial until used for preparation of the microscope specimens. Further dilutions were made later in the vials as found to be necessary. Because of the large dilutions required, the heterogeneous nature of the materials studied, and the high magnifications obtained, very careful controls were maintained throughout all the procedures in order to prevent contamination. To prepare specimens for the electron microscope, a very thin film of Formvar (polyvinyl formal) was cast on a clean water surface and transferred to a one-eighth-inch disk of 200-mesh, stainless-steel screen. A drop of the sample suspension was placed on the plastic film and allowed to settle and dry in an air-conditioned room (72° F., 60 percent relative humidity). At this stage, the specimen was ready for examination.

Shadowcasting Technique

The RCA type EMU (50 kv) electron microscope was used. The direct magnification of this instrument ranges from about 1,000 to 25,000 diameters, with a possible useful magnification of about 250,000 diameters obtainable through optical enlargement of the micrographs. In this study it was found most suitable to use about 6,000 diameters as the original electronic magnification, with subsequent optical enlargement to give a total linear magnification of about 21,000. The 1μ line on the photographs illustrating this report represents 1 micron. In most cases, at least five 2-inch-square micrographs were made of characteristic fields of each material examined. In all, about 1,250 electron micrographs were taken.

Since metallic shadowcasting (*23, 24*) adds contrast as well as a three-dimensional aspect to surface details, a very thin coating of evaporated chromium metal was deposited obliquely upon the dispersed and mounted specimens. This was accomplished by placing the dried specimens in a vacuum chamber at a fixed angle and distance from a tungsten-wire basket containing about 75 mg. of pure chromium. When the pressure in the vacuum chamber reached 10^{-5} mm. of mercury, the tungsten crucible was electrically heated, causing chromium vapor to radiate in all directions. As the atoms condensed, a very thin film of metal was deposited on the specimens. The sample particles are elevations behind which are shielded areas (shadows) receiving no metallic coating. In the microscope, such areas readily transmit the incident beam of electrons, whereas the thicker and chromium-coated areas scatter the electron beam and less of the incident energy is transmitted.

Thus, from the length of the "bright" shadows, as observed in the electron microscope, relative heights of the particles and details of surface structure can be deduced. Positive photographic prints are shown in all of the illustrations in order to permit direct comparison between shadowed and unshadowed specimens. These represent the images as seen in the microscope. Black indicates material opaque to the electron beam; white indicates transparent material. Shadow length is approximately four times the altitude of the particles.

Results and Discussion

Electron micrographs of several soils and clay minerals are presented in figures 2–7. These show only a few of the many samples examined, and were selected to illustrate some of the more evident points of information available through the use of the electron microscope. In a random scanning of a specimen, marked differences are encountered, as illustrated in the two micrographs of a sample of Manor clay (figs. 7*A* and 7*B*) and those of Illinoian glacial drift (figs. 7*C* and 7*D*). Thus, a single micrograph or view, as used in this report, is not necessarily representative of the whole material under discussion but is, however, of value in pointing out certain features of the sample. On the other hand, in the actual examination a rapid survey of the whole specimen, or usually of several specimens, permits a quick comparison of the general morphology of the material and a rough estimate of the relative proportions of different constituents, even though they have not been fully identified. Where possible, the most representative field was chosen for the micrograph.

Kaolins

In soils work, when clays are mentioned, the kaolins first come to mind. In fact, clay formation has long been referred to as "kaolinization." This is the result of the early location and use of kaolin deposits and the belief that kaolin was the chief constituent of soil clays. Although kaolin may be the predominant clay mineral present in some soils, it is now understood that it may be only a minor component or entirely absent in other soils. Originally assumed to be a single mineral, it is now recognized as a family or group of clay minerals, of which kaolinite is the leading member.

Crystals of a well-formed kaolinite are shown in figures 2*A* and 2*B* as thin, six-sided plates. The perfection of crystal outline varied considerably among the several kaolinites examined—in some cases, only an occasional straight side or 120-degree angle could be found, although the thin, platelike characteristic was common to all specimens. It may be noted that some of the crystals are elongated, whereas the faces of others are more nearly equidimensional. Stacking and thinness of the plates are evident, both from their transparency and from the length of shadows. Most of the plates lie in the plane of the plastic substrate but the "book" of kaolinite crystals, in the upper part of figure 2*A*, illustrates an aggregate with the plates nearly at right angles to the film. The relatively great depth of field of the electron microscope is demonstrated in areas of figure 2*A*, where settling of kaolinite particles upon each other during the drying of the specimen has superimposed several plates of various sizes and thicknesses.

This feature is of exceptional value in making visible the details of a wide range of particle sizes and thicknesses at a single focusing adjustment. These relatively pure kaolinites, English china clay, and Georgia kaolin, were formed as residual deposits from granite, apparently under conditions favorable to good crystalization. On the other hand, most of the commercial kaolin clays examined were much less perfectly formed and relatively less pure.

Halloysite, seen in figures 2*C* and 2*D*, is characterized by tubular-shaped crystals, as indicated by the greater transparency of their hollow centers. Split tubes and concentric layering may also be observed. In earlier examinations of halloysite, the particles were thought to be lathlike (*2*), but later studies confirmed the hollow, tubular nature (*3, 5, 6, 9*). This crystal form is in sharp contrast to the six-sided plates of kaolinite, despite the fact that both minerals have the same chemical composition, approaching $Al_2O_3 \cdot 2SiO_2 \cdot 2H_2O$. Although the existence of cylindrical characteristics has been revealed, the exact mechanism of the growth of tubular crystals has not been explained. Nevertheless, the marked differences in the morphologies of these two kaolins must have a profound influence on the physical properties of soils containing these minerals.

With regard to the other members of the kaolin family, dickite showed a platy habit very similar to that of kaolinite, and allophane consisted of irregular, angular masses with a glassy appearance. Nacrite was not examined in this study. It has been reported to resemble kaolinite and dickite in form, but with particle sizes as large as 5 mm. (*3, 9*).

Although micrographs of allophane have been published (*9*) in which the particles were shown as spherical masses, our observations were not in agreement with this. We noted that the particles were angular at the start but, after a short exposure to the electron beam, were sufficiently melted to produce the globular shapes. Compared to halloysite and kaolinite, the minerals dickite, nacrite, and allophane are relatively rare and are not thought to occur widely in soils.

Montmorillonites

Although the identity of montmorillonite as a mineral was established many years ago, it was not at first regarded as being a part of soil clays. More recently, however, it has been shown to be widespread in soils. The term montmorillonite is now used to designate the clay-mineral family as well as its chief member. Among the other members are nontronite, beidellite, and hectorite. The familiar trade term bentonite has been applied to adsorptive clays and drilling muds but properly refers to a clay rock consisting essentially of the sodium-rich montmorillonite or, less often, beidellite clay mineral. The origin of bentonite is considered to stem from the devitrification and chemical alteration of glassy igneous material. In soils, however, the montmorillonites may be either water laid or formed residually from a variety of parent minerals.

As shown in figures 2*E*, 2*F*, and 3*A–D*, the appearance of the members of the montmorillonite family is in decided contrast to that of the kaolins. The montmorillonite member (fig. 2*E*) is characterized by loose, fluffy floccules or aggregates of indefinite shape and size. The nebulous outlines and the variable translucency indicate aggregates formed from much smaller and thinner units. These features seem to be correlated with the magnitude of surface area required for the high base-exchange capacity, large volume changes, and the strong bonding and plastic properties manifested by the montmorillonites. Beidellite (fig. 3*B*) is similar in appearance to the montmorillonite member, as are the bentonites (figs. 2*F* and 3*A*). Nontronite, the iron-rich member, is seen in figure 3*C* as long, thin fibers which have a tendency to remain in bundle-like aggregates, or sometimes as long, flat, ribbon shapes. Hectorite, (fig. 3*D*) is in the form of very thin needles or fibers, shorter than those of nontronite, with a pronounced tendency towards a more random arrangement. When not completely dispersed, hectorite crystals are arrayed in thin, lath-like aggregates.

The crystal lattice structure of the montmorillonites would indicate the occurrence of platy particles. Although montmorillonite, beidellite, and the bentonites are platy,

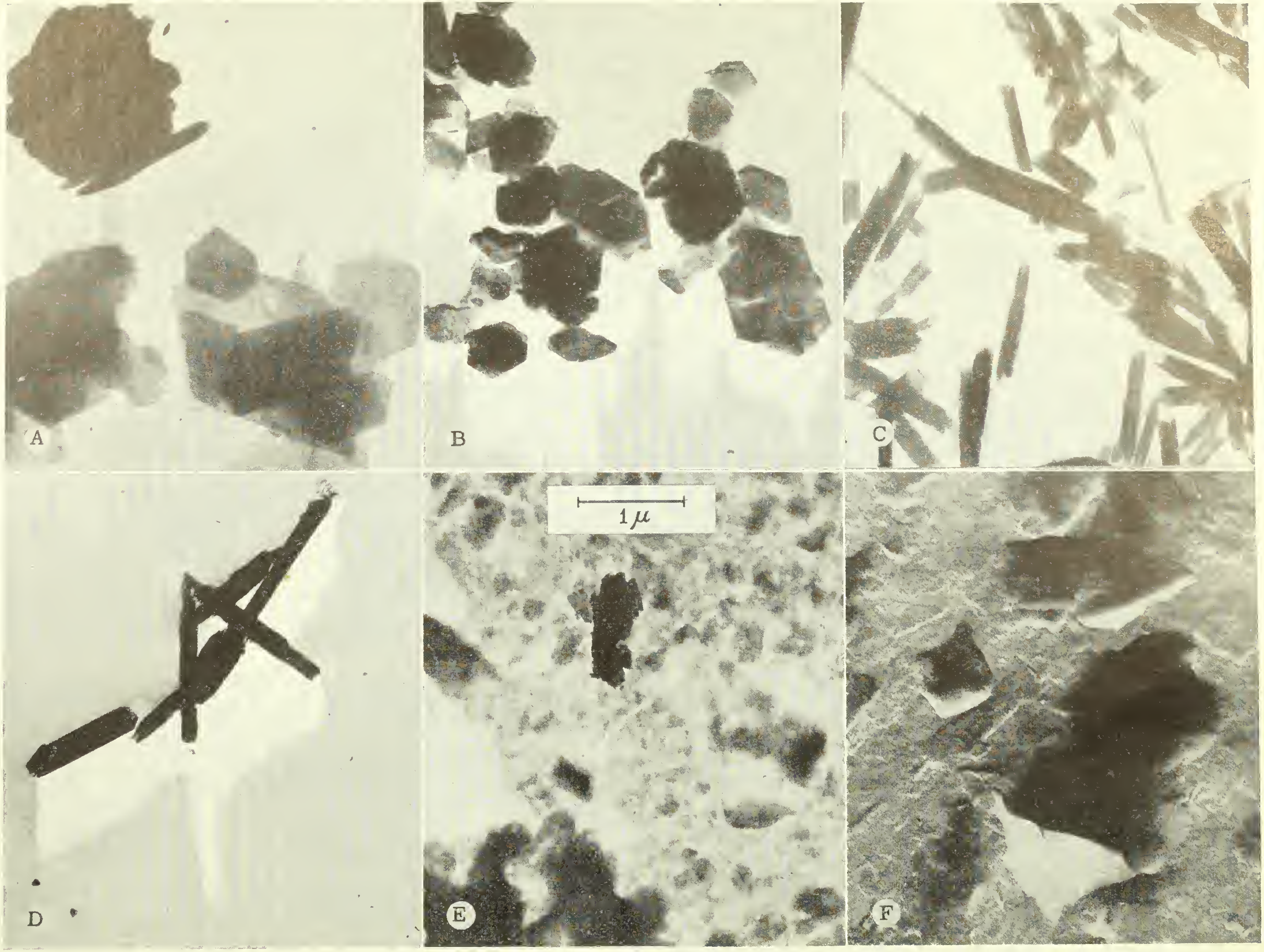

Figure 2.—A, English china clay (kaolinite); B, Georgia kaolin; C, D, halloysite, Utah Lake, Utah; E, montmorillonite, Otay Mesa, Calif.; F, sodium bentonite, Wyoming (electron micrographs: total electronic and optical magnification = 21,000 diameters).

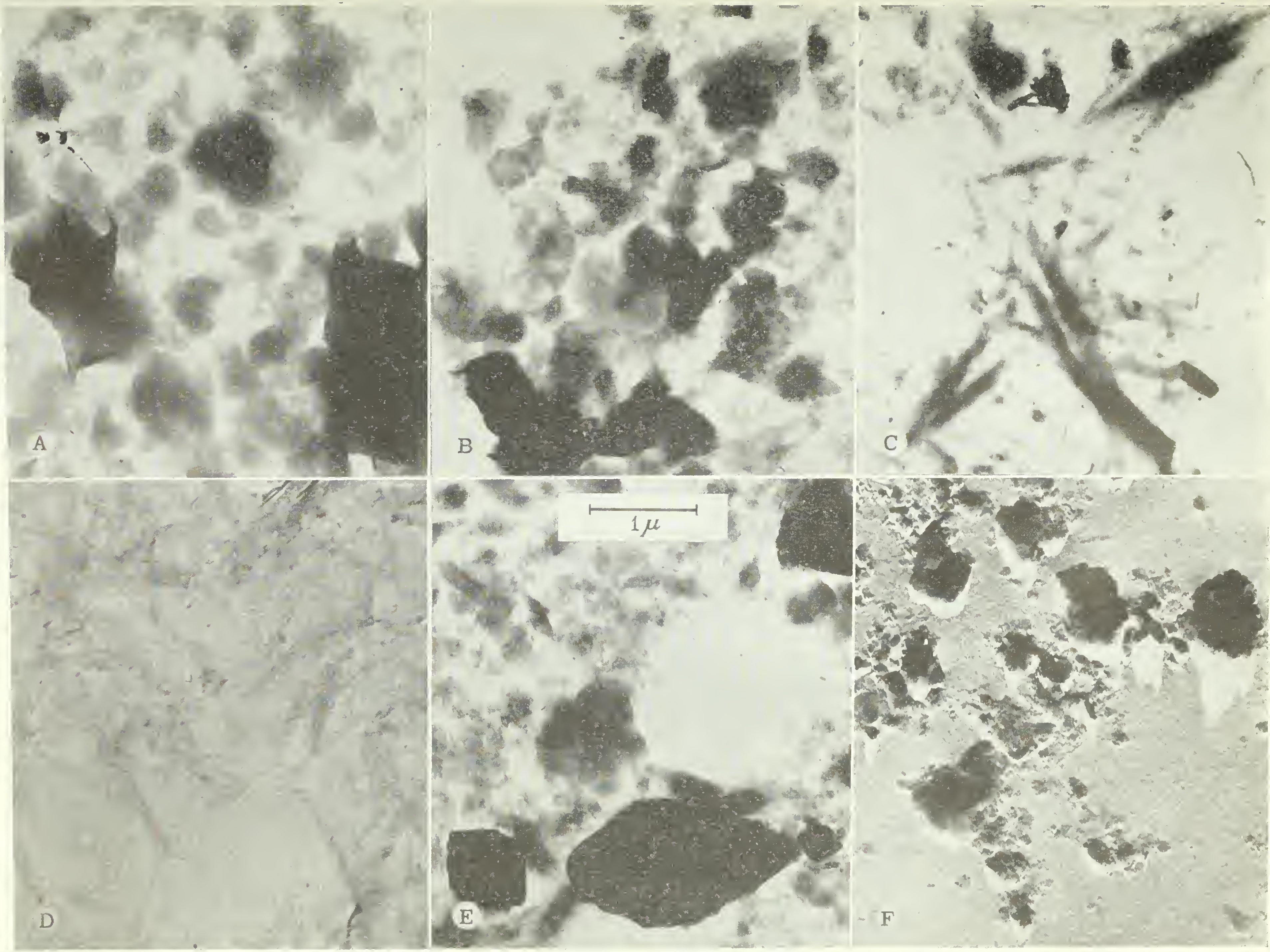

Figure 3.—A, calcium bentonite, Mississippi; B, beidellite, San Diego County, Calif.; C, nontronite, Manito, Wash.; D, hectorite, Hector, Calif.; E, illite, Morris, Ill.; F, illite, Fithian, Ill. (electron micrographs: total electronic and optical magnification = 21,000 diameters).

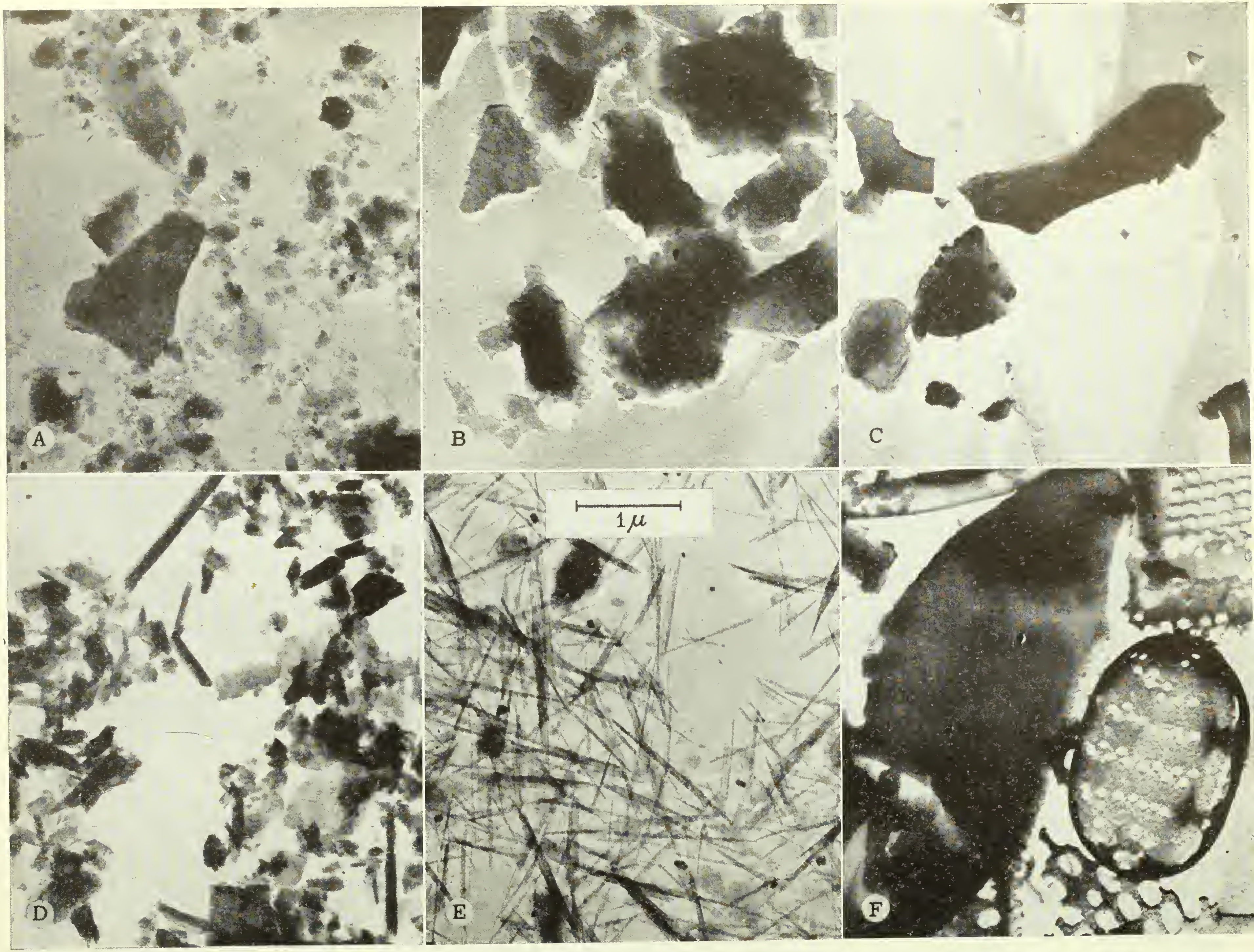

Figure 4.—A, illite, Fithian, Ill.; B, metabentonite, Tazewell, Va.; C, silica (quartz), Ottawa, Ill.; D, mica, Spruce Pine, N. C.; E, attapulgite, Attapulgus, Ga.; F, diatoms, Santa Barbara County, Calif. (electron micrographs: total electronic and optical magnification = 21,000 diameters).

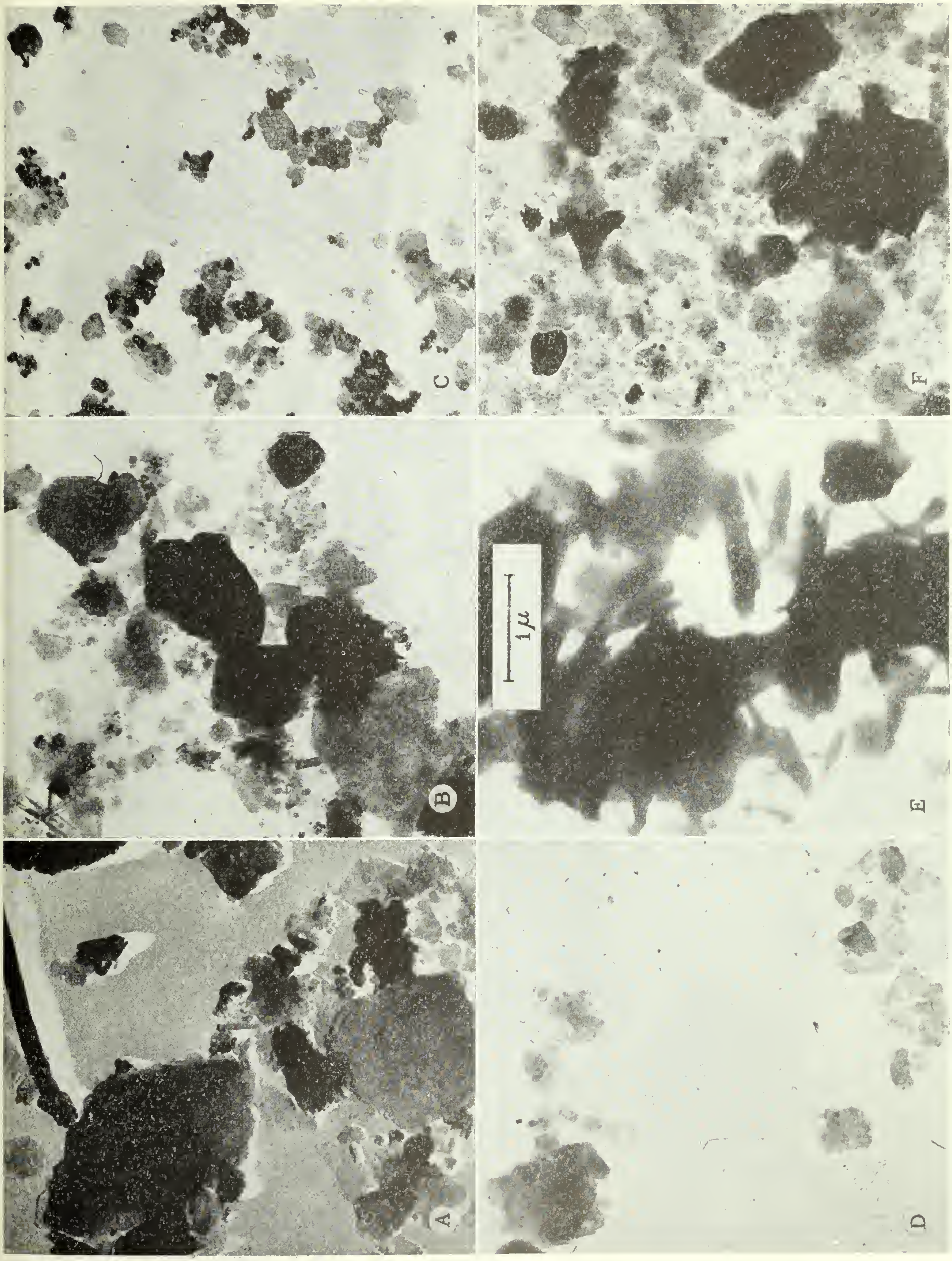

Figure 5.—A, Cecil clay; B, Tuxedo clay; C, Philadelphia red clay; D, Philadelphia white clay; E, Houston clay; F, Putnam clay (electron micrographs: total electronic and optical magnification = 21,000 diameters).

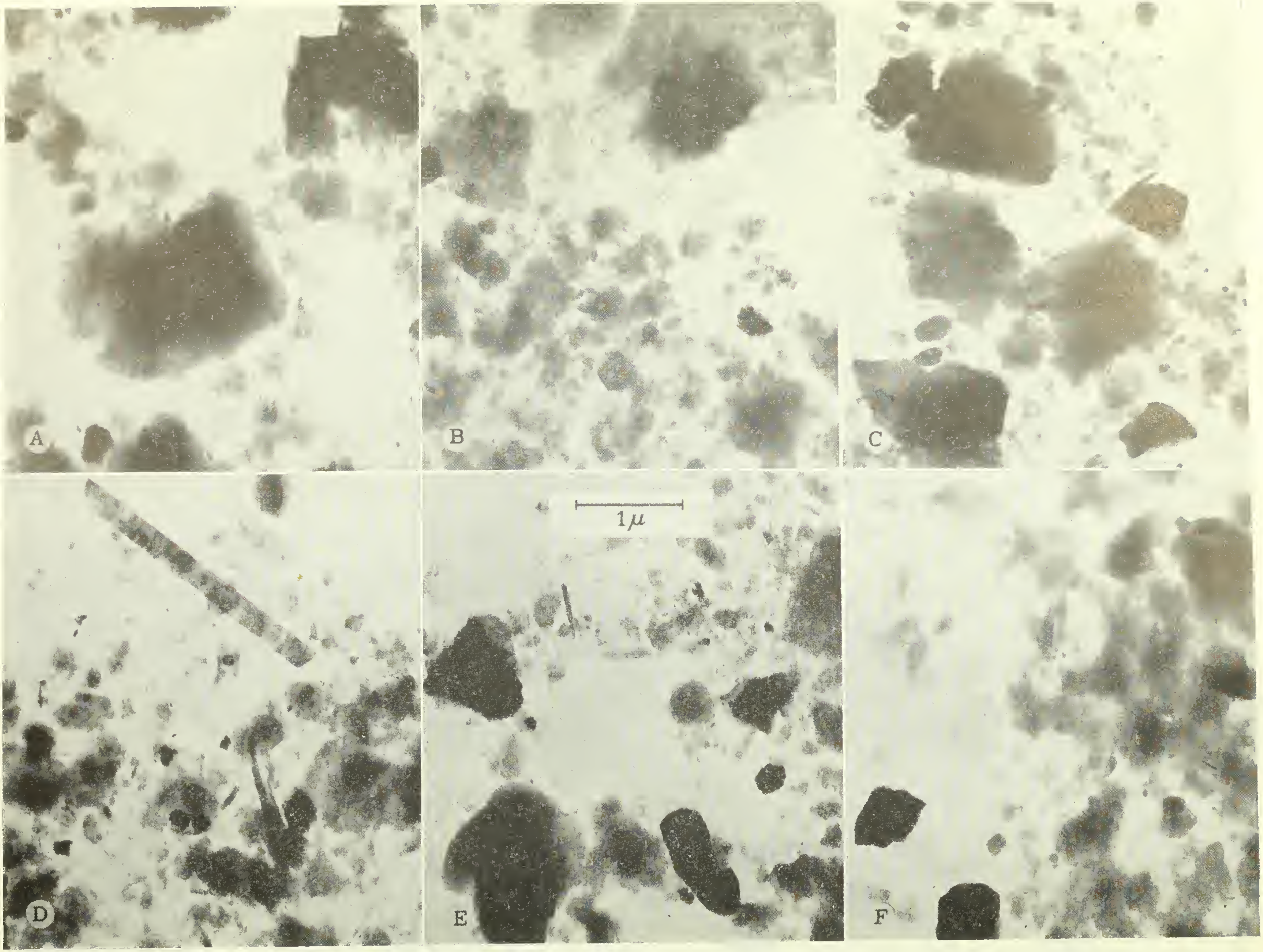

Figure 6.—A, Iredell clay; B, Miami clay; C, Crosby clay; D, Gila clay; E, Hagerstown clay; F, Wabash clay (electron micrographs: total electronic and optical magnification = 21,000 diameters).

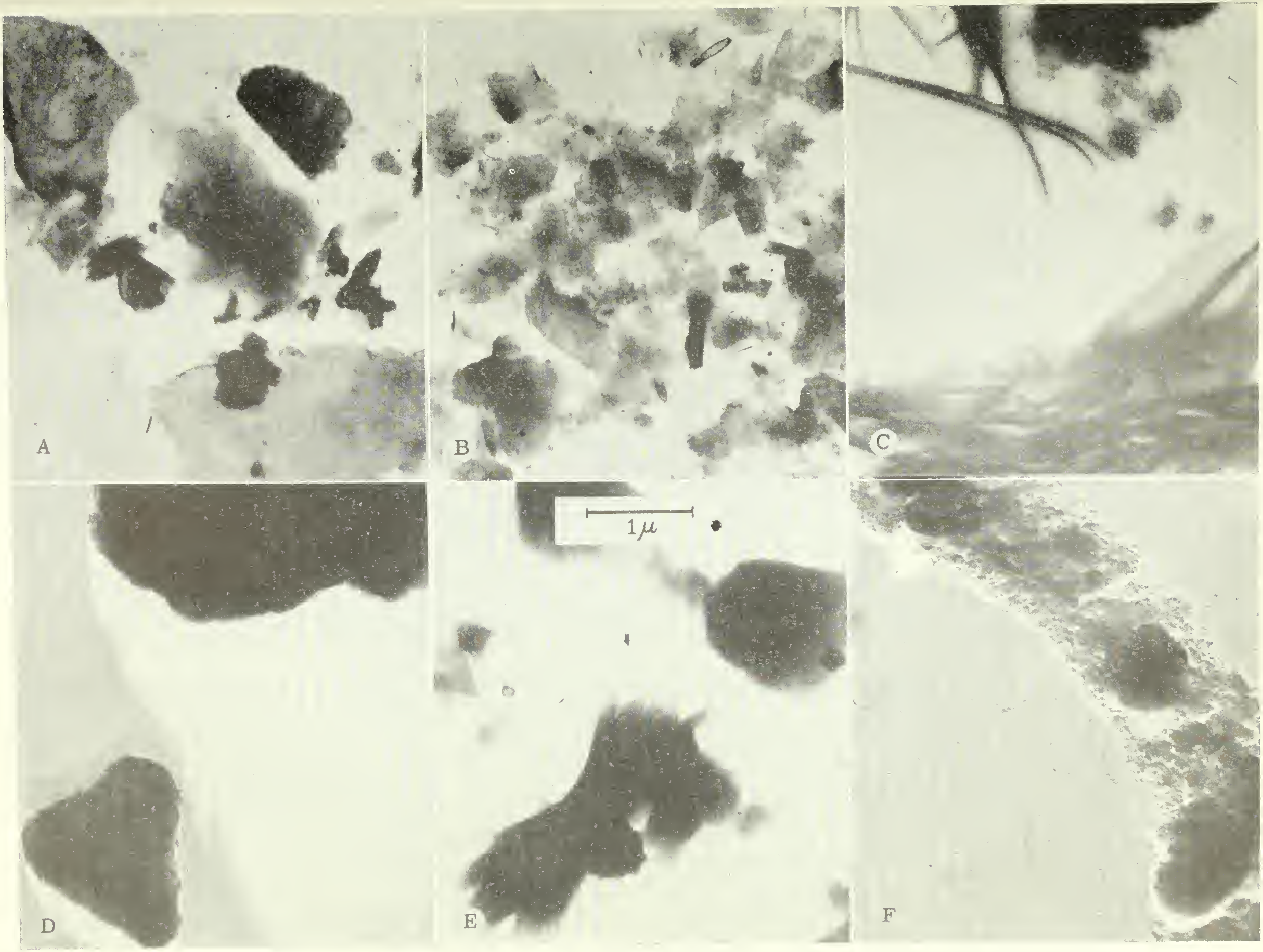

Figure 7.—A, B, Manor clay; C, D, clay from Illinoian glacial drift; E, Marshall clay; F, Pullman clay (electron micrographs: total electronic and optical magnification = 21,000 diameters).

this is not the case with nontronite and hectorite which exhibit a crystal habit resembling needles or fibers. The occurrence of nontronite in the ribbon-like form may be attributed to the greater atomic radius of the ferric ion in the crystal lattice, resulting in a strain restricting the width but permitting growth along the length to produce fibrous crystals (*21*). The long narrow crystals of hectorite are so thin and transparent as to be almost indistinguishable individually.

Illites

The illite or hydrous mica group of clay minerals is not as well known and understood in some respects as are the two groups previously discussed. The name illite is used as "a general term for the clay-mineral constituents of argillaceous sediment belonging to the mica group" (*11*); specific characterization into separate species within the group is still to be accomplished. Illite is reported as a common constituent of slates, shales, and soil clays (*4, 11, 12*).

As seen in figures 3*E*, 3*F*, and 4*A*, illite particles are vague and indefinite in shape, with little resemblance to crystalline material. However, plates or flakes with rough but relatively sharp boundaries may be observed. It may be noted that some of the illite material closely resembles the fluffy habit shown by montmorillonite. This feature makes difficult, if not impossible, a positive distinction between these two materials. In general, however, illite presents a denser and more definitive appearance than montmorillonite. Flakes with relatively sharper outlines and frequently of fairly large diameter are also more common to illite. These are most likely remnants or relics of partly altered mica. The two euhedral masses seen at the bottom of figure 3*E* are thought to be iron pyrite; the large irregular mass between them is probably quartz.

Related or Allied Minerals

Many of the non-clay minerals likely to be associated with soil clays were observed to bear little resemblance to any of the clay minerals. In general, they were dense masses with no distinct morphologies. However, some of them, such as mica, attapulgite, and diatomaceous earth, have distinctive features and are easily recognized. Particles of ground silica (fig. 4*C*) were, as expected, quite fragmental, with sharp points and knife-like edges. The platy habit of finely divided mica is seen in figure 4*D*. Although this sample was represented as a very pure mica, it contained a considerable amount of a rod-shaped impurity in addition to the mica sheets. These were different from the narrow strips which mica yields as a result of secondary cleavage perpendicular to the plane of the sheets, as shown at the top of figure 6*D*. It may be noted that some of the thin, nearly transparent mica sheets in figure 4*D* show uneven bands resulting from interference or diffraction phenomena between the layers. During the manipulation of the microscope controls to bring various fields of the specimen into focus, these bands frequently were seen to move across the mica sheets. This movement was thought to be the result of heating and electrical charging, with the effect of driving off water from between the sheets or buckling of the sheets themselves. This latter effect, although useful in the examination, could not be photographed.

The acicular structure of attapulgite, a fuller's earth from Attapulgus, Ga., is seen in figure 4*E*. Although, in general, this needle-like form appears similar to that of sepiolite and serpentine, the crystals of attapulgite are characteristically shorter and straighter than those of other fibrous minerals.

Part of a sample of a diatomaceous earth is shown in figure 4*F*. This material consists chiefly of the siliceous remains of diatoms, a large class of microscopic algae which contribute to deposits derived from both fresh and sea water.

Examination of Soil Materials

The clay minerals are for the most part the chief constituents of soil clays. It has been clearly recognized that the kind and amount of clay in a soil strongly affect its suitability as an engineering material and thus have a direct bearing on the design, construction, and maintenance of highways and other structures. The foregoing discussion and the interpretations and concepts derived from the electron micrographs of the pure clay minerals cannot, however, be expected to delineate fully the nature of soil clays. The genesis of soils is a dynamic process, characterized by a colloidal system involving a large number of free ions and the hydrous oxides of silicon, aluminum, and iron. Varied parent material, weathering conditions, and dynamic organic processes, along with the vigorous operations of dislodgement, transportation, and deposition, combine to produce a soil environment that, in general, makes the soil clay a heterogeneous body with chemical and physical properties quite at variance with those of any single, pure, well-formed clay mineral. A study of the mineral content of soil clays, however, in conjunction with studies of the clay minerals themselves, provides a logical approach to the complex mechanisms relating such factors as origin, chemical alteration, crystalline structure, and variable constitution of these principal constituents whose physical and chemical behavior are so strongly reflected in the properties of the soil body.

A selection of electron micrographs of several of the soils examined is presented in figures 5–7. As previously noted, these do not comprise a comprehensive coverage of all possible soil conditions, but are presented as a random view of what may be seen in the examination of a selection of soils from many different areas and backgrounds of soil formation. The mixed nature of soil clays is a point immediately brought out by most of the micrographs, in direct contrast to the relatively greater purity of the mineralogical specimens. For the most part, the tubes of halloysite remain relatively unchanged in form, as do also other rod or needle-like materials, but well-formed crystals of kaolinite as six-sided plates are almost entirely absent.

Soils Containing Kaolinite

Examples of soils in which kaolinite is the predominant clay mineral are given in the micrographs of Cecil (fig. 5*A*) and Tuxedo (fig. 5*B*) subsoils and of Philadelphia red clay (fig. 5*C*). Cecil is a residual soil formed from granitic rocks in the southern part of the Piedmont soil province, just west of the Atlantic Coastal Plain. Tuxedo and Philadelphia red clay are water-laid, sedimentary materials common to the Coastal Plain itself. The latter, taken from a deep boring in Philadelphia, was well-mottled with white areas, a common effect among poorly drained subsoils. Parts of the white areas were used to prepare the micrograph, in figure 5*D*, of Philadelphia white clay. In these several figures, kaolinite appears as the major component, with an admixture of opague (dark) rounded material.

In the natural state, each of these soils is strongly colored by red oxides of iron, probably hematite, which may be the opaque material seen in the micrographs. The mottling apparently is the result of abrupt changes in the iron-oxide content as the white clay contains very little of this ingredient and is mostly pure kaolinite.

Most of the particles seen in these samples are quite distinct in outline, with sharp, although irregular, edges. In the case of Tuxedo clay (fig. 5*B*) there is, however, a considerable quantity of small-grained, poorly defined material which may provide a clue to the reasons for the known highly plastic nature of this clay, since kaolinite alone is usually found to be only weakly plastic. Although electron microscopy does not provide an absolute identification of this material, the value of such an examination is clearly demonstrated in the indication of its presence so that the aid of other methods may be applied to its further study and possible identification.

Other Soil-Clay Constituents

Montmorillonites have been well-established (*1, 13, 21*) as common constituents of soil clays in varying proportions to other clay minerals. Of the several soils examined in this study, examples in which montmorillonites are predominant are shown in figures 5*E*, 5*F*, and 6*A*. These include Houston clay (*8, 20*), from the Gulf Coastal Plain of Texas; Putnam (*22*) from a loessial area of Missouri; and Iredell (*21*), a residual soil derived from diabasic rocks in the southern Piedmont. Iredell (fig. 6*A*) appears to consist largely of a single, relatively pure substance closely resembling the nebulous floccules of the mineral specimens of montmorillonite.

Putnam, shown in figure 5*F*, contains beidellite (*21*, *22*)—very similar in appearance to the montmorillonite member. In addition, there is a non-clay constituent ranging in size down to a small fraction of a micron. It may be noted here that both magnesium-saturated and natural Putnam soils were used in this work, as were also examples of other soils having sodium and calcium as the adsorbed cations, but that little difference in morphology, except perhaps in the mode of flocculation, could be ascribed to the difference in cations. In the case of the sodium and calcium bentonites shown in figures 2*F* and 3*A*, respectively, the floccules exhibit warping and resemble a cornflake shape.

Examples of Miami, Crosby, Gila, and Hagerstown soils, reported (*1*, *7*, *8*) to be high in illite content, are shown in figures 6*B*, 6*C*, 6*D*, and 6*E*. Miami (fig. 6*B*) is widespread in the North Central States and is derived from calcareous glacial till. The transparent, smaller particles are believed to be illite in association with darker fragments of non-clay minerals. Some of the larger particles may also be illite or aggregates of illite and montmorillonite, which is also present (*7*) in Miami soils. The presence of rod and needle-like crystals and many small dark fragments, in figure 6*B*, illustrate the mixed composition not unexpected in a material derived from glacial action.

A comparison of figure 6*B*, of Miami clay, and figure 6*C*, of Crosby clay, shows the close similarity of the clays of these two soils, which are both members of the Miami soil catena and are derived from the same parent calcareous glacial till under different conditions of slope and drainage.

This is of interest in that electron-microscopial examination may assist in the correlation of soil types and in pointing out similarities or differences between two examples of the same soil obtained from different areas. For instance, the colloidal content of these two soils from Indiana has been reported (*7*) to consist chiefly of hydrous mica (illite) with a smaller content of montmorillonite. Although their mineralogical compositions were closely similar, they were different in thermal decomposition and base-exchange behavior. Other work on the clay of a Miami soil from Wisconsin (*8*) found montmorillonite predominant and illite as a minor constituent. In cases of this sort, a more detailed study of specific separations of the various constituents might prove helpful.

Gila clay, from a desert soil of Arizona, derived from granitic and gneissic rocks, is seen in figure 6*D* to contain an abundance of illite (*8*) as well as a rich admixture of non-clay minerals. Note the strip of mica in the upper-central portion of the micrograph. Hagerstown clay (fig. 6*E*) also appears to consist largely of illite. This soil is formed residually in limestone areas of the Eastern and Central States. Samples of Hagerstown clay from Maryland and Missouri have been reported (*1*) to contain about equal proportions of kaolinite and illite. The sample used here, obtained from Monroe County in Indiana, apparently contains both of these along with other minerals.

Supporting Information Needed

It is believed that montmorillonite and illite may be associated in mixed layer aggregates (9). The metabentonite from Tazewell, Va., shown in figure 4*B* may be a clay mineral of this sort. Such occurrences are even more probable in soil clays, as in Wabash clay (fig. 6*F*), an alluvial soil reported to contain both of these minerals. Such intimately mixed, layered aggregates of extremely fine-grained particles cannot be clearly differentiated and identified by electron microscopy alone.

The need for supporting information is especially evident where opaque masses largely lacking in distinctive features are present, as well as where some soil-clay constituents are unidentifiable yet have more specific morphologies. For example, the fibrous material in figure 7*C*, resembling sepiolite, was observed in a sample of Illinoian glacial drift but cannot be positively identified from the micrographs alone. Sheaves of very fine fibers, possibly nontronite, were found in the clay extracted from Marshall silt loam, a prairie soil from the loessial area of western Iowa (fig. 7*E*) as well as in Houston clay (fig. 5*E*). A gel-like material, possibly silica gel, is shown in figure 7*F*, covering and binding together the more discrete particles of Pullman clay, a soil occurring extensively in the Texas Panhandle. The effect of such gels on the properties of soils is probably of considerable importance but as yet has not been evaluated. Certainly, however, their presence is discernible only by a method permitting direct visual observation in the colloidal range. Soils derived from mica-bearing parent material may also contain micas in the clay fraction. Similarity of the platy habit of mica and kaolinite make their differentiation difficult when the crystals are poorly developed, small in size, and fragmented and aggregated, as in Hagerstown clay (fig. 6*E*).

Summary

In the study of materials so small in particle size, complex in nature and properties, and varied in composition as soil clays, a single method cannot possibly provide a complete understanding of the many factors involved. The use and value of X-ray diffraction, differential thermal analysis, and chemical-analytical approaches are well-established. Electron microscopy can be very helpful in identifying and characterizing some of the constituents of soil clays and in providing a rough estimate of their relative proportions. For constituents not possessing specific morphological features, it can provide complementary clues and a background for information obtained from other methods.

Particle size and the relative distribution of particle-size ranges are well-known to be important in considerations of soil properties. These are no less important in the case of the soil-clay fraction itself. In a correlation of the amount of clay or of its quality to the behavior of a soil, separation must be made at some arbitrary figure for the upper particle-size limit. In estimating the effect of different sized particles on the behavior of the soil, however, consideration must be given to the particle sizes found in the whole soil sample as well as in the clay fractions obtained therefrom. In the clay size range, such considerations are helped in a direct way by electron microscopy, in which particle size and shape can be observed and semiquantitative estimates of the relative amounts of size ranges may be made.

Having once examined a variety of soil clays with the electron microscope, one is convinced that any detailed investigation of similar materials by other methods alone would be severely handicapped for want of visual examination. Moreover, the application of surface-replica techniques and the further development of other methods of specimen preparation will likely be of value in studying the structure of both consolidated and natural soils and the effect of additives in the amelioration or control of soil properties. In its own right, the electron microscope provides valuable information about soil clays which can be obtained in no other way. Admittedly, this essential concept has been well recognized by those closely allied to this research tool. It was felt, however, that the methods of electron microscopy should be specifically directed to the problems associated with soils as engineering materials. And it is from this point of view that this preliminary report has been made.

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A Cooperative Study of
Fillers in Asphaltic Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

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The judicious use of local aggregates and of mineral byproducts in highway construction can effect considerable savings in costs, through lower purchase prices and lesser transportation charges. Large waste piles of traprock fines, accumulated at quarries, and fly ash, a residue from combustion of powdered coal, offer cheap sources of fines for use in bituminous surfacing if they are found to be satisfactory for the purpose.

The cooperative study reported here was initiated to determine, by laboratory test, an evaluation of such materials as fillers in asphaltic concrete. In this type of pavement, a satisfactory filler aids in resisting the softening action of water on the asphalt-aggregate mixture. It was found, in these tests that, for the materials tested, traprock dust would give satisfactory, and fly ash superior, resistance to water in bituminous concrete paving mixtures of the dense type. Other effects of these fillers were not investigated, and it is possible, of course, that they may be of some importance.

From this study it was found that the 4-day, 120° F. immersion of the standard immersion-compression test generally can be relied upon to differentiate between satisfactory and unsatisfactory fillers. For border-line cases, a 7- or 14-day immersion may be necessary to clear up any doubt.

Where determination of compressive strength or stability is important, the study showed that test specimens must be molded from freshly prepared mixtures, without any reheating or reprocessing. This restriction is apparently unnecessary when the only information desired is the effect of water on the compressed mixture.

ONE of the important problems that requires the attention of highway, materials, and research engineers is that of evaluating the mineral aggregates that may be potentially available for use in highway base and surface construction. The judicious use of local aggregates and of various byproducts is an important means of reducing construction costs.

In Massachusetts, the quarrying and processing of large quantities of traprock for various purposes, including highway construction, has resulted in the accumulation of large waste piles of traprock fines which, so far, have not been marketable and therefore constitute an expensive and troublesome disposal problem.

In the hope that a use might be found for at least a part of this material and with a view to reducing bituminous surfacing costs to some extent, a cooperative research project[1] was initiated to determine, by laboratory tests, an indication of the possible value of traprock dust as a filler for asphaltic concrete. Investigation was also made of the value of fly-ash fillers, obtained as a byproduct from the combustion of powdered coal.

It was agreed that the evaluation of the proposed fillers should be made by using them in a standardized asphaltic concrete which would be prepared, molded into test specimens, cured, and tested under carefully controlled standard conditions and procedures.

Conclusions

The first two of the following conclusions relate to the resistance to water of mixtures containing the fillers under investigation. Attention is called to the fact that other possible effects of the fillers were not investigated and it is possible that, in some cases, these may be of importance.

The results of this laboratory study have been consistent in indicating that traprock dust of the type furnished by New Jersey and Massachusetts can be expected to give satisfactory resistance to water when used as a filler in bituminous concrete paving mixtures of the dense type.

The laboratory studies indicated consistently that the fly-ash fillers, represented by those from New Jersey and one from Illinois, can be expected to give superior resistance to water in bituminous concrete paving mixtures of the dense type.

The tentative standard immersion-compression test,[2] requiring immersion at 120° F. for 4 days, generally can be relied upon to differentiate between satisfactory and unsatisfactory fillers. For obtaining this differentiation, the criterion of 75-percent retention of the dry strength, which was suggested in connection with earlier studies, appears to have been further substantiated by the data in this report since the fillers that failed to meet it were those that, on the basis of reported experience, would be expected to give poor results in the field. However, further correlation with field experience seems desirable to establish more firmly the lowest permissible percentage of retained strength.

In cases where fillers, although passing the 4-day immersion test, are suspect because they are border-line cases or because of high silica content, extension of the immersion period to 7 or 14 days may be resorted to in order to clear up any doubt.

Where measurement of the compressive strength or stability of the mixture is the main consideration, it is important that the test specimens be molded from freshly prepared mixtures without reheating or reprocessing of any kind, since it has been found that both reheating and reworking—as, for instance, breaking up pavement samples and remixing—cause substantial increases in the apparent compressive strength.

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[1] The cooperators were the laboratories of the Massachusetts Department of Public Works, represented by A. V. Bratt, Chief of Laboratory, who originally proposed the study; the New Jersey State Highway Department, represented by Fred H. Baumann, Chief, Testing Division; the Ohio State Department of Highways, represented by R. R. Litehiser, Chief Engineer, Testing and Research Laboratory; the National Crushed Stone Association, represented by A. T. Goldbeck, Engineering Director; and the Bureau of Public Roads, represented by E. F. Kelley, Chief, Physical Research Branch. This report has been reviewed and approved by all the cooperators.

[2] Tentative method of test for effect of water on cohesion of compacted bituminous mixtures. A.S.T.M. designation D 1075-49T.

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When the only information desired is the effect of water on the compressed mixture, it appears from past experience and from the data in this report that the caution against reheating or reworking need not be observed because the increase in strength caused by these factors is approximately proportional in the dry and wet tests and, therefore, the percentage of strength retained is not appreciably affected.

Basis of Study

The principal test characteristic upon which the ratings of the fillers were based was the resistance of the compacted asphaltic concrete mixture to loss of strength after immersion in water at 120° F. for 4 days. The procedure for making this test, commonly called the immersion-compression test, was that outlined in a paper before the Association of Asphalt Paving Technologists[3] and later adopted in two parts as tentative standards[4] by the American Society for Testing Materials. Briefly, the test for a given mixture consists of subjecting a set of three companion specimens of the compacted mixture to an unconfined compression test to determine the "dry" compressive strength. A duplicate set of three specimens, prepared at the same time as the first, is immersed in water for 4 days at 120° F. and this set is then tested for "wet" compressive strength. The percentage of the original or "dry" compressive strength that is retained in the "wet" test is reported as the retained strength. With only such exceptions as will be noted in connection with the test data, the specimens for both the dry-strength test and the immersion test, in these studies, were cured in air for 24 hours at 140° F. after molding as required in the standard procedure.

Crushed traprock for coarse aggregate, and glacial sand for fine aggregate, as well as two filler materials—traprock dust and commercial limestone dust—were furnished for the first series of tests by the Massachusetts laboratory.

At about the time the initial work was getting under way, the suggestion was made that a similar study of fly ash, a waste product of coal-fired, electric power plants, be made a part of the project[5]. The laboratories of Massachusetts, New Jersey, Ohio, and the Bureau of Public Roads agreed to take part in this second phase of the investigation.

Additional quantities of crushed traprock and glacial sand, as well as a sample of silica dust, were supplied by the Massachusetts laboratory and two samples of filler, one a silica dust and the other a fly ash, were supplied by the New Jersey laboratory. Silica dust was included as a check or control material because of its known, generally poor, behavior as a filler.

Upon completion of the second phase of the study, a third series of tests was proposed in which sufficient new samples of traprock dust and fly ash were to be tested to corroborate fully the indications obtained in the two previous series. An additional coarse aggregate, rhyolite from Massachusetts, was brought into the study and rhyolite dust made from this same material was also included, together with two traprock dusts and three additional fly-ash samples that were supplied by the New Jersey laboratory. Other coarse aggregates and fillers were included in the work of some of the individual laboratories as will be indicated by the data in the tables and discussions.

Master Grading Used

In the original plans, a master grading was worked out and was used throughout the three phases of the study. Because of variations in the apparent specific gravity of the different materials used, the only way that a constant particle-size distribution could be maintained was to reduce all of the proportions in the master grading to an absolute-volume basis. For this purpose, the absolute volume of 7 parts by weight of limestone dust per 100 parts of total aggregate was taken as the basis for all the filler contents throughout the three series of tests. Thus, the variations in the amount by weight of filler shown in the various mixtures that will be described reflect only the variations in the apparent specific gravity of these fillers. Similarly, when coarse aggregate other than the original crushed stone was used, the proportion by weight was adjusted to give the same absolute volume of coarse aggregate as before.

In the first series, the bitumen content was 6½ parts by weight to 100 parts by weight of basic aggregate. In the second series, some of the mixtures contained 6½ parts and some 5½ parts of asphalt per 100 parts of aggregate and in the third series, 5½ parts of asphalt were used for all mixtures with two exceptions which will be discussed in connection with the tabulated data.

It was originally intended that the test mixtures should contain between 6 and 7 percent voids when compacted in order that the variable effect of water due to differences in the filler might be clearly defined. It was found, however, that the type of filler affected the density and, therefore, the void content, to a very considerable degree. No attempt was made to adjust out these differences because it was felt that they reflected some of the inherent differences in the fillers that should be brought out in the test results.

The basic grading that was used throughout the filler investigation is shown in table 1, which also shows the asphalt ratio that was used in the first series and part of the second.

The proportions by weight of the two test mixtures used in the first series of tests are shown in table 2. The mixture containing traprock dust shows the adjustment in the weight of the filler to obtain the correct absolute volume. The mixture containing limestone dust is, of course, the basic mixture shown in detail in table 1.

Table 1.—Basic grading and mixture proportions for mixture containing limestone dust

	Parts by weight			
	Traprock	Sand	Limestone dust	Combined
Sieve sizes:				
½ inch to No. 4	38			38
No. 4 to No. 10	20	5		25
No. 10 to No. 40		19		19
No. 40 to No. 200		11		11
No. 200			7	7
Total aggregate	58	35	7	100
Asphalt, 85–100 penetration				6.5
Total, parts by weight				106.5

Table 2.—Proportions of the test mixtures in series 1

	Parts by weight of mixture with filler of—	
	Limestone dust	Traprock dust
Aggregate retained on No. 200 sieve	93	93
Filler	7	7.48
Total aggregate	100	100.48
Asphalt	6.5	6.50
Total	106.5	106.98

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[3] *Application and present status of the immersion-compression test*, by J. T. Pauls and J. F. Goode, 1947 Proceedings of the Association of Asphalt Paving Technologists. See also, *Further developments and application of the immersion-compression test*, by the same authors, PUBLIC ROADS, vol. 25, No. 6, Dec. 1948.

[4] A.S.T.M. *Standards*, 1949, Part 3. Test methods D 1074-49T and D 1075-49T.

[5] This phase of the study was suggested by Mr. Fred H. Baumann, Chief, Testing Division, New Jersey State Highway Department.

Table 3.—Proportions of the test mixtures in series 2

	Parts by weight of mixture with filler of—					
	Limestone dust [1]	Traprock dust [1]	Silica dust			Fly ash (N. J.)
			Md.	Mass.	N. J.	
Aggregate retained on No. 200 sieve	93	93	93	93	93	93
Filler	7	7.48	6.70	6.87	6.75	6.57
Total aggregate	100	100.48	99.70	99.87	99.75	99.57
Asphalt, series 2a	6.5	6.5	6.5	6.5	6.5	6.5
Mixture totals, series 2a	106.5	106.98	106.20	106.37	106.25	106.07
Asphalt, series 2b	5.5	5.5	5.5	5.5	5.5	5.5
Mixture totals, series 2b	105.5	105.98	105.20	105.37	105.25	105.07

[1] Same fillers as used in series 1.

The proportions by weight of the test mixtures of the second series are shown in table 3.

In planning the third series of tests, it was agreed that each cooperating laboratory would carry out only as many of the tests as could be done without interference with other work, and also that some of the laboratories might properly diverge from the group effort to conduct special studies on materials primarily of local interest. Thus, a great many material combinations were tested in the third series but relatively few of these were tested by more than one of the cooperators. A list of the material combinations of series 3 that were investigated by one or more of the cooperating laboratories is given in table 4. One of the cooperating laboratories made several additional mixtures which were used in exploring the effect of other methods of curing than those agreed upon and used by the group as a whole. The data from these extra tests are not reported here because there is no basis upon which they can be compared with the data obtained by the other cooperating laboratories.

Traprock Dust a Good Filler

The results of the tests on the two materials of series 1 are shown in table 5. Here the comparison is only between one sample of limestone dust and one sample of Massachusetts traprock dust. The four laboratories that participated in this first series showed good agreement on the percentage of original strength retained by the test mixtures after immersion. All of the results agreed in indicating that, as determined by this test, the traprock dust was equal to limestone dust.

The results of the cooperative tests on the mixtures of series 2 are shown in table 6. The New Jersey silica dust, a material already existing in the deposit as a natural powder and produced from the natural deposit primarily for the glass industry, was reported to have given good results as a filler in bituminous surfacing. This would seem to make it an unusual example of silica dust which, as a rule, is not favored as a filler on the basis of its service record. This material showed good performance in the laboratory tests of series 2a, with high bitumen content. In series 2b, with low bitumen content, one sample gave good results and one gave poor results.

The Maryland silica dust, while showing good performance in the high-bitumen mixtures (6½ parts), was definitely inferior in those containing 5½ parts of bitumen per 100 parts of aggregate.

At the conclusion of the series 1 tests, it had been suggested that the bitumen content of 6½ parts might be high enough to mask the effect of poor quality in certain fillers and it had been agreed that two pro-

Table 4.—Material combinations tested in series 3[1]

Mixture No.	Coarse aggregate		Fine aggregate		Filler (dust)[1]	
	Type	Source	Type	Source	Type	Source
1	Traprock	Mass	Glacial sand	Mass	Traprock E	N. J.
2	do	do	do	do	Traprock F	Do.
3	do	do	do	do	Rhyolite	Mass.
4	do	do	do	do	Traprock	Do.
5	do	do	do	do	Fly ash A	N. J.
6	do	do	do	do	Fly ash B	Do.
7	do	do	do	do	Fly ash C	Do.
8	do	do	do	do	Fly ash	Ill.
9	Rhyolite	do	do	do	Traprock E	N. J.
10	do	do	do	do	Traprock F	Do.
11	do	do	do	do	Rhyolite	Mass.
12	do	do	do	do	Traprock	Do.
13	do	do	do	do	Fly ash A	N. J.
14	do	do	do	do	Fly ash B	Do.
15	do	do	do	do	Fly ash C	Do.
16	do	do	do	do	Fly ash	Ill.
17	do	do	Potomac River sand	D. C	do	Do.
18	do	do	do	do	Rhyolite	Mass.
19	do	do	do	do	Mica dust	Md.
20	Blast furnace slag	Md	do	do	Fly ash	Ill.
21	do	do	do	do	Rhyolite	Mass.
22	do	do	do	do	Mica dust	Md.
23[2]	Traprock	Mass	Glacial sand	Mass	Traprock	Mass.
24	do	do	do	do	Limestone	Do.
25[3]	do	do	do	do	Traprock	Do.
26	do	do	do	do	Limestone	Do.
27	Traprock E	N. J	do	do	Traprock E	N. J.
28	do	do	do	do	Limestone	Mass.
29	Traprock	Va	do	do	Traprock	Va.
30	do	do	do	do	Limestone	Mass.
31	Quartzite	Pa	do	do	Quartzite	Pa.
32	do	do	do	do	Limestone	Mass.
33[4]	Rhyolite	Mass	do	do	Rhyolite	Do.
34	do	do	do	do	Limestone	Do.
35	Traprock E	N. J	do	do	do	N. J.
36	do	do	do	do	Traprock E	Do.
37	do	do	do	do	Traprock F	Do.
38	do	do	do	do	Silica	Do.
39	do	do	do	do	Fly ash A	Do.
40	do	do	do	do	Fly ash B	Do.
41	do	do	do	do	Fly ash C	Do.
42	do	do	do	do	Fly ash D	Do.
43	Traprock F	do	do	do	Limestone	Do.
44	do	do	do	do	Traprock E	Do.
45	do	do	do	do	Traprock F	Do.
46	do	do	do	do	Silica	Do.
46x[5]	do	do	do	do	do	Do.
47	do	do	do	do	Fly ash A	Do.
48	do	do	do	do	Fly ash B	Do.
48x[6]	do	do	do	do	do	Do.
49	do	do	do	do	Fly ash C	Do.
50	do	do	do	do	Fly ash D	Do.
51	Traprock	Mass	do	do	Traprock E	Do.
52	do	do	do	do	Traprock F	Do.
53	do	do	do	do	Fly ash A	Do.
54	do	do	do	do	Fly ash B	Do.
55	do	do	do	do	Fly ash C	Do.
56	do	do	do	do	Rhyolite	Mass.
57	do	do	do	do	Limestone	Do.
58	do	do	do	do	Traprock	Do.
59	Rhyolite	do	do	do	Rhyolite	Do.
60	do	do	do	do	Limestone	Do.
61	do	do	do	do	Traprock	Do.

[1] All fillers in mixtures 1–22, 25, and 26 sieved to pass No. 200 sieve. Mixtures 20, 21, and 22 contained about 2 parts of extra asphalt to satisfy the absorption of the slag, making the total asphalt content 7½ parts, but their effective asphalt content may be considered to be 5½ parts. Mixtures 56–61 contained 6 parts of asphalt per 100 parts of aggregate (basic mix basis) as compared to 5.5 parts for the others reported in series 3.

[2] Same as No. 4, except filler not all passing No. 200 sieve.

[3] Same as No. 4.

[4] Same as No. 11, except filler not all passing No. 200 sieve.

[5] This is a rerun of a duplicate sample of No. 46.

[6] This is a rerun of a duplicate sample of No. 48.

Table 5.—Results of the immersion-compression test on the mixtures of series 1[1]

Testing laboratory	Reported voids	Strength		
		Dry	Wet	Retained
LIMESTONE DUST MIXTURES				
	Percent	*P.s.i.*	*P.s.i.*	*Percent*
A	3.2	310	309	100
B		316	292	92
C	2.5	259	235	91
D	5.5	259	242	93
TRAPROCK DUST MIXTURES				
A	3.5	292	302	100+
B		326	306	94
C	3.1	252	229	91
D	5.7	264	246	93

[1] All mixtures contained 6½ parts of asphalt per 100 parts of aggregate, the basic mix basis. The fillers were sieved through the No. 200 sieve and only the passing portion was used. Both fillers were furnished to all the cooperators by the Massachusetts laboratory.

Table 6.—Results of the immersion-compression test on the mixtures of series 2

Testing laboratory	Filler	Source of filler	Reported voids	Strength		
				Dry	Wet	Retained
SERIES 2a.—ASPHALT CONTENT 6½ PARTS PER 100 PARTS AGGREGATE						
			Percent	*P.s.i.*	*P.s.i.*	*Percent*
D	Limestone dust	Mass	4.4	263	239	[1] 91
C	Silica dust	N. J.	3.2	236	203	[2] 86
D	do	do	3.9	251	213	85
C	do	Md	3.9	234	212	91
D	do	do	5.5	252	204	81
D	do	do	4.5	252	198	[3] 79
D	do	Mass	5.8	243	199	82
D	Fly ash	N. J	3.6	212	212	100
SERIES 2b.—ASPHALT CONTENT 5½ PARTS PER 100 PARTS AGGREGATE						
B	Limestone dust	Mass	5.9	353	283	80
D	do	do	8.2	256	225	88
B	Traprock dust	do	5.6	352	283	80
D	do	do	8.5	245	227	93
B	Silica dust	Md	6.4	367	246	67
B	do	N. J	6.3	317	278	88
D	do	Md	8.4	261	144	55
D	do	N. J	6.9	281	189	67
B	Fly ash	do	3.8	321	299	93
D	do	do	6.5	224	226	100

[1] Check test. First-run test result (see table 5) was 93 percent retained.
[2] Extending the 120° F. immersion to 7 days reduced this to 68 percent.
[3] Check test. First-run test result (see preceding line in table) was 81 percent retained.

portions, 6½ parts and 5½ parts of bitumen, would be used in the series 2 tests and that some silica dusts would be included to explore this factor. The results of the tests of series 2 indicated that the high bitumen content did, in fact, have the anticipated masking effect and it was decided that, in the mixtures for test series 3, only 5½ parts of bitumen would be used.

In series 2, as in series 1, the traprock dust performed at least as well in the laboratory test as the limestone dust. The average values of retained strength for the mixtures of series 2b were 86+ percent for traprock dust and 84 percent for limestone dust. The one sample of fly ash then available appeared to be superior to limestone dust. The retained strengths for this sample reported by laboratories B and D, series 2b, averaged 96+ percent.

Supplemental Studies

In addition to the mixtures listed in table 6, a number of additional mixtures were made and tested by variously modified procedures to develop information concerning questions that occurred to the several cooperators during the course of the main investigation. By means of such auxiliary studies, laboratory B noted that when only the portion passing the No. 200 sieve was used, both the limestone dust and the traprock dust of series 1 gave slightly better results than when the material was used as received with a small percentage retained on the No. 200 sieve. Although not particularly significant, the differences show a tendency, that has been believed to exist, for the coarser portion of so-called filler materials to fail to function as filler unless the aggregate as a whole happens to be deficient in intermediate fines. The comparisons are shown in table 7.

Effect of Extended Immersion and Reprocessing

In other auxiliary tests, laboratory C showed that silica dust, even in a rich mixture, was distinctly sensitive to extended immersion at 120° F., the retained strength after immersion dropping from 86 percent at 4 days to 68 percent at 7 days (table 6, footnote 2). These tests also showed that the same silica-dust mixture was also damaged somewhat more by immersion for 4 days at 140° F. than at 120° F., while the traprock-dust and limestone-dust mixtures apparently were not adversely affected by the higher immersion temperature. These relations are shown in table 8. It should be pointed out, however, that the mixtures used in this study of the effect of increased immersion temperature were mixtures that had been molded, cured, and tested, then reheated to 300° F., broken down or "disintegrated," remolded, and again cured for the tests at the increased immersion temperature. For this reason, the effect of temperature of immersion alone should not be

Table 7.—Comparative effect of fillers used as received and as sieved to pass the No. 200 sieve

Mixtures with—	Retained strength after 4 days immersion at 120° F.	
	Filler all passing No. 200	Filler used as received
	Percent	*Percent*
Limestone dust	92	91
Traprock dust	94	91

Table 8.—Effect of reheating and remolding previously compacted specimens of bituminous concrete

Mixtures with—	Strength					
	Dry		Wet		Retained	
	Original	Reheated, remolded	Original, immersed at 120° F.	Reheated, remolded, immersed at 140° F.	Original, immersed at 120° F.	Reheated, remolded, immersed at 140° F.
	P.s.i.	*P.s.i.*	*P.s.i.*	*P.s.i.*	*Percent*	*Percent*
Limestone dust	259	366	235	366	91	100
Traprock dust	252	363	229	328	91	90
Silica dust	236	347	203	274	86	79
Average	249	359	222	323	89	90
Average increase due to reprocessing, percent		44		45		

Table 9.—Results of immersion-compression tests on plant mixtures reheated in the oven to 350° F. to prepare them for molding

Mixtures with—	Strength		
	Dry	Wet	Retained
	P.s.i.	*P.s.i.*	*Percent*
Limestone dust	582	517	89
Traprock dust	731	655	90

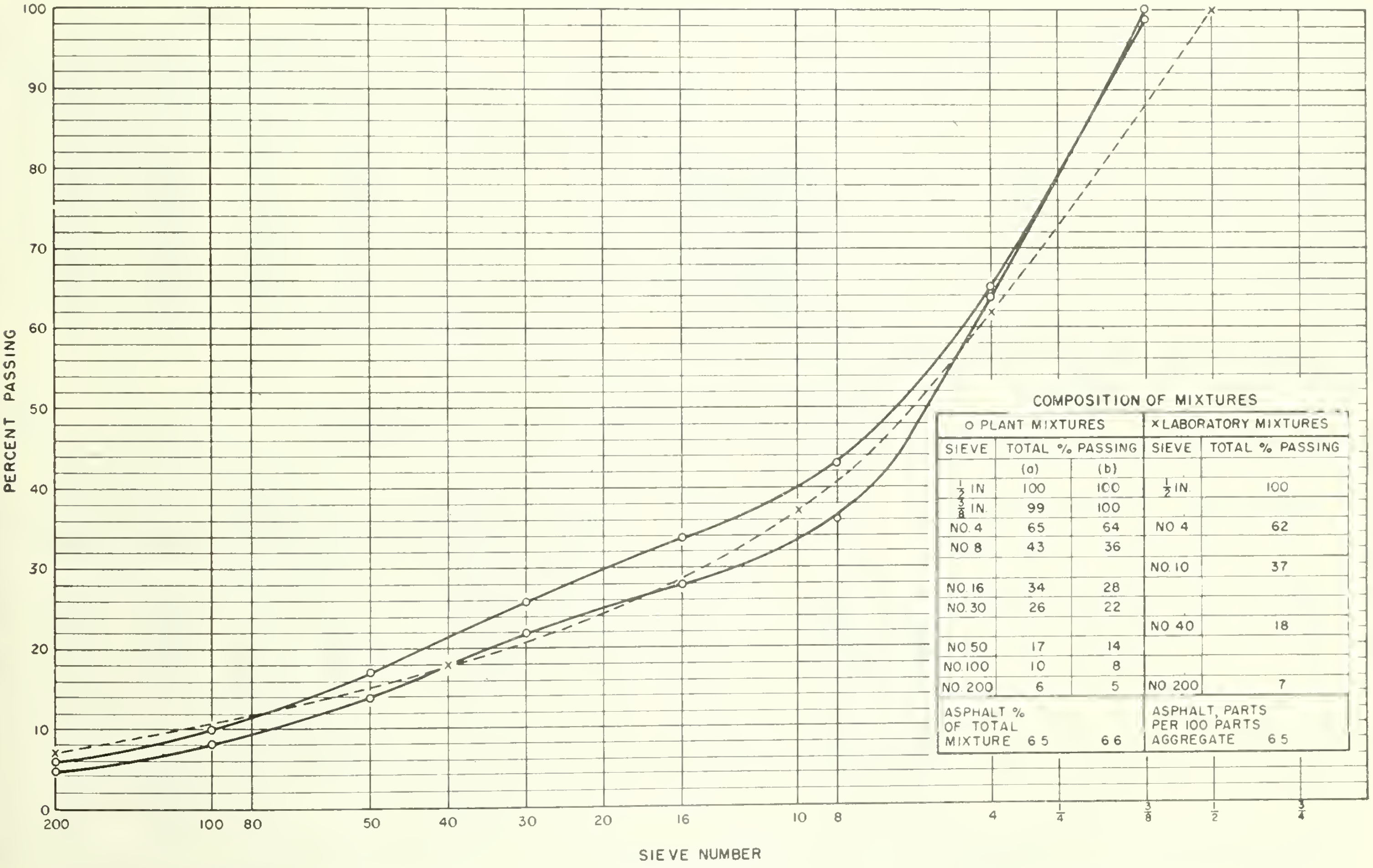
considered conclusive. Of considerably more importance and value is a comparison of the unit compressive strengths shown by these mixtures before and after reheating and remolding. This comparison is also shown in table 8.

The data in table 8 show that the overall effect of reprocessing these previously compacted specimens was to increase the average dry strength by 44 percent and the average wet strength by 45 percent. This corroborates numerous previous observations made in connection with the testing of bituminous mixtures. These observations have also indicated that where only reheating of cooled, uncompacted mixtures is necessary in order to mold test specimens, the compressive strength values are considerably higher than for specimens molded from freshly prepared mixtures that have not been allowed to cool below a suitable molding temperature.

Tests by laboratory B on two large samples of plant mix, prepared by a paving contractor and shipped to the laboratory for test, showed the effect of reheating a cold, loose mixture and possibly other factors that could not be isolated. The compressive strengths and the results of the immersion-compression test on these mixtures are shown in table 9.

One of these two mixtures contained limestone dust and the other traprock dust. The gradings of the extracted aggregates are shown in figure 1, the small circles representing the plotting points for these two mixtures. The basic grading for the laboratory mixtures used in the cooperative study is also shown in figure 1, the points being indicated by X. From the curves, it is apparent that the gradings of the two commercial mixtures were very similar to each other and to that of the laboratory mixtures except for slightly lower dust content in the commercial mixtures. The grade of asphalt in the commercial mixtures was not known, but is believed to have been the same as that used in the laboratory mixtures. The mixtures were, of course, molded and tested some considerable time after they were made. In spite of the probable effect of difference in age and perhaps other factors, however, the strikingly high compressive strengths of the test specimens from these two mixtures are believed to reflect primarily the effect of reheating in connection with molding. Comparison of these compressive strengths, shown in table 9, may be made with those of corresponding laboratory mixtures that were molded while hot immediately after mixing, shown in table 5.

Auxiliary tests similar to those reported in table 5 were made by laboratory A. The only change in procedure was to increase the water-bath temperature for the 4-day immersion from 120° F. to 140° F. For this set of tests, where reheating and remolding were not involved, the increased immersion temperature resulted in a decrease in wet compressive strength and



COMPOSITION OF MIXTURES

O PLANT MIXTURES			X LABORATORY MIXTURES	
SIEVE	TOTAL % PASSING		SIEVE	TOTAL % PASSING
	(a)	(b)		
1/2 IN	100	100	1/2 IN.	100
3/8 IN.	99	100		
NO. 4	65	64	NO 4	62
NO 8	43	36		
			NO. 10	37
NO. 16	34	28		
NO. 30	26	22		
			NO 40	18
NO 50	17	14		
NO. 100	10	8		
NO. 200	6	5	NO 200	7
ASPHALT % OF TOTAL MIXTURE	6 5	6 6	ASPHALT, PARTS PER 100 PARTS AGGREGATE	6 5

Figure 1.—Comparison of basic laboratory mixture with plant mixtures tested by laboratory B.

percentage of retained strength for both the limestone-dust and traprock-dust mixtures. This comparison is shown in table 10.

Table 10.—Effect of changing the immersion temperature from 120° F. to 140° F. (immersion-compression test, immersion period 4 days)

Mixtures with—	Strength: Immersion at 120° F. Dry	Immersion at 120° F. Wet	Immersion at 120° F. Retained	Immersion at 140° F. Dry	Immersion at 140° F. Wet	Immersion at 140° F. Retained
	P.s.i.	*P.s.i.*	*Percent*	*P.s.i.*	*P.s.i.*	*Percent*
Limestone dust	310	309	100	310	283	91
Traprock dust	292	302	100+	299	257	86

Series 3 Tests

The results of the tests by the five cooperating laboratories on the mixtures of series 3 are shown in tables 11–16. All of the mixtures of series 3, except those tested by laboratory A, contained 5½ parts of asphalt per 100 parts of aggregate. Laboratory A used 6 parts per 100. Most of the cooperating laboratories used, in addition to the regular immersion for 4 days at 120° F., one or more variations of the immersion procedure. This was done in order to develop a broader knowledge of the effect of various test conditions. The study was also expanded in series 3 to include several additional coarse aggregates, one additional sand, and several additional samples of fly ash and other fillers.

Table 11 shows the results of the tests of series 3 made by laboratory B. This table provides, in the main, a comparison between traprock dust and limestone dust as filler material, with various coarse aggregates and one type of glacial sand from Massachusetts. The two tests, involving quartzite dust in one case and rhyolite dust in the other, are auxiliary tests, as were the tests on silica dust throughout the study. The results of these tests should not be compared directly with those of series 1 and the first part of series 2 because of the lower bitumen content of the series 3 mixtures. On the whole, the traprock dusts were indicated to be as resistant to water action in the mixtures as the limestone dust. In the standard exposure test, immersion at 120° F. for 4 days, the lowest retention for a traprock-dust mixture was 85 percent and for a limestone-dust mixture, 89 percent. The high values were 94 percent for traprock and 92 for limestone. The averages were 89+ for traprock dust and 90 for limestone dust.

It is interesting to note the small variation in percentage of retained strength for the limestone-dust mixtures, considering the variety of coarse aggregates used. This confirms the results of other studies having to do with the application of the immersion-compression test in which it was noted that the detrimental effect of somewhat inferior coarse aggregate could be compensated by using, among other things, a high-quality filler. There is considerably more variation between the high and low values of retained strength for the traprock-dust mixtures, probably indicating some variation in the quality of the traprock dust from Massachusetts, Virginia, and New Jersey, as well as the effect of different coarse aggregates.

The high retention of strength of the quartzite-aggregate mixture with quartzite filler is interesting and somewhat surprising

Table 11.—Results of series 3 immersion-compression tests [1] by laboratory B

Coarse aggregate	Fine aggregate	Filler	Air voids	Dry strength	Immersed 4 days: Wet Strength	Immersed 4 days: Retained strength	Immersed 14 days: Wet Strength	Immersed 14 days: Retained strength
			Percent	*P.s.i.*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
Traprock (Mass.)	Glacial sand	Traprock (Mass.)	6.8	301	258	86	227	75
do	do	Limestone dust	7.0	310	281	91	267	86
do	do	Traprock (Mass.)[2]	6.1	324	275	85	230	71
do	do	Limestone dust [2]	6.3	305	280	92	274	90
Traprock E (N. J.)	do	Traprock E (N. J.)	6.5	293	276	94	242	83
do	do	Limestone dust	6.6	311	279	90	269	86
Traprock (Va.)	do	Traprock (Va.)	4.2	363	339	93	305	84
do	do	Limestone dust	5.2	330	294	89	281	85
Quartzite (Pa.)	do	Quartzite (Pa.)	4.1	375	363	97	322	86
do	do	Limestone dust	4.4	352	314	89	308	88
Rhyolite (Mass)	do	Rhyolite (Mass.)	4.3	309	163	53	55	18
do	do	Limestone dust	4.6	304	271	89	257	85

[1] All mixtures contained 5.5 parts 85–100 penetration asphalt per 100 parts of aggregate. Immersion temperature 120° F.
[2] Filler, 100 percent passing No. 200 sieve.

Table 12.—Results of series 3 immersion-compression tests [1] by laboratory C

Coarse aggregate	Fine aggregate	Filler	Air voids	Dry strength	Immersed 4 days: Wet Strength	Immersed 4 days: Retained strength	Immersed 7 days: Wet Strength	Immersed 7 days: Retained strength	Immersed 14 days: Wet Strength	Immersed 14 days: Retained strength
			Percent	*P.s.i.*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
Traprock E (N. J.)	Glacial sand (Mass.)	Limestone dust	5.5	297	235	79	228	77	217	73
do	do	Traprock E (N. J.)	5.8	290	223	77	213	73	181	62
do	do	Traprock F (N. J.)	6.0	304	236	78	211	69	196	64
do	do	Silica dust (N. J.)	5.3	277	225	81	119	43	48	17
do	do	Fly ash A (N. J.)	4.3	235	226	96	242	100+	222	94
do	do	Fly ash B (N. J.)	3.5	262	230	88	225	86	224	85
do	do	Fly ash C (N. J.)	3.7	243	246	100+	229	94	215	88
do	do	Fly ash D (N. J.)	3.2	244	222	91	235	96	225	92
Traprock F (N. J.)	do	Limestone dust	5.0	360	277	77	268	74	262	73
do	do	Traprock E (N. J.)	4.8	329	283	86	275	84	257	78
do	do	Traprock F (N. J.)	5.6	313	275	88	250	80	235	75
do	do	Silica dust (N. J.)	5.3	334	222	66	52	16	38	11
do	do	Fly ash A (N. J.)	4.0	291	277	95	272	93	267	92
do	do	Fly ash B (N. J.)	3.5	257	286	100+	277	100+	270	100+
do	do	Fly ash C (N. J.)	3.6	334	285	85	271	81	272	81
do	do	Fly ash D (N. J.)	3.2	276	278	100	283	100+	260	94

[1] All specimens tamped 25 blows on each of two layers before compressive load applied. All mixtures contained 5.5 parts 85–100 penetration asphalt per 100 parts of aggregate. Immersion temperature 120° F.

and, together with the test results on the mixtures containing New Jersey silica dust in series 2, may indicate that not all silica-dust fillers should be assumed to be unfit for use in bituminous concrete. It is important to remember, however, that final conclusions should not be drawn from laboratory results alone but should be subject to confirmation by observation of performance in field service.

Immersion Period Extended

Increasing the immersion period from 4 days to 14 days generally had a minor effect on the percentage of strength retention except in the case of the rhyolite-dust mixture, which dropped from 53 percent at 4 days to 18 percent at 14 days. In this case, the increased immersion period did not change the estimate of the value of the rhyolite filler because it would have been considered unsuitable on the basis of its 53 percent strength retention after the 4-day immersion test.

Table 12 shows the results of the tests of series 3 made by laboratory C. The groups of mixtures in the upper and lower halves of the tabulation are the same ex-

Table 13.—Results of series 3 immersion-compression tests [1] by laboratory E

Coarse aggregate	Fine aggregate	Filler	Air voids	Strength		
				Dry	Wet	Retained
			Percent	*P.s.i.*	*P.s.i.*	*Percent*
Traprock (Mass.)	Glacial sand (Mass.)	Traprock E (N. J.)	9.5	339	263	78
do	do	Traprock F (N. J.)	9.9	355	271	76
do	do	Fly ash A (N. J.)	7.2	297	278	94
do	do	Fly ash B (N. J.)	7.4	315	271	86
do	do	Fly ash C (N. J.)	7.5	316	273	86

[1] All mixtures contained 5.5 parts 85–100 penetration asphalt per 100 parts of aggregate. Immersion for 4 days at 120° F.

Table 14.—Partial results of series 3 immersion-compression tests [1] by laboratory A

Coarse aggregate	Fine aggregate	Filler	Air voids	Strength		
				Dry	Wet	Retained
			Percent	*P.s.i.*	*P.s.i.*	*Percent*
Traprock (Mass.)	Glacial sand (Mass.)	Rhyolite (Mass.)	7.0	400	253	63
do	do	Limestone dust	7.0	414	288	70
do	do	Traprock (Mass.)	7.7	378	277	73
Rhyolite (Mass.)	do	Rhyolite (Mass.)	3.9	324	232	72
do	do	Limestone dust	2.9	333	361	100+
do	do	Traprock (Mass.)	2.9	332	345	100+

[1] Six parts 85–100 penetration asphalt per 100 parts of aggregate used in all mixtures. All specimens in this group cured by the standard method, 24 hours at 140° F. Immersion for 14 days at 120° F.

Table 15.—Partial results of series 3 immersion-compression tests [1] by laboratory A

Test	Aggregate			Air voids	Special dry strength after additional curing 14 days at 120° F.	Immersed 4 days at 120° F. after additional curing 10 days at 120 °F.		Immersed 14 days at 120° F. after standard curing only. (24 hours at 140°F.)	
	Coarse	Fine	Filler			Wet Strength	Retained strength	Wet Strength	Retained strength
				Percent	*P.s.i.*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
A	Traprock (Mass.)	Glacial sand	Rhyolite (Mass.)	7.0	428	376	88	253	59
B	do	do	Limestone dust	7.0	447	399	89	288	64
C	do	do	Traprock (Mass.)	7.7	433	393	91	277	64
	Rhyolite (Mass.)	do	Rhyolite (Mass.)	3.9	427	397	93	232	54
	do	do	Limestone dust	2.9	443	406	92	361	81
	do	do	Traprock (Mass.)	2.9	427	413	97	345	81
X	Traprock (Mass.)	do	Rhyolite (Mass.)	8.5	630	533	85	357	57
Y	do	do	Limestone dust	7.7	600	544	91	397	66
Z	do	do	Traprock (Mass.)	8.5	560	514	92	354	63

[1] Six parts 85–100 penetration asphalt per 100 parts of aggregate used in all mixtures. All specimens in this group cured by the standard method, 24 hours at 140° F. Some were then given additional curing as indicated in column heads. Tests X, Y, and Z are check runs on A, B, and C.

Table 16.—Results of series 3 immersion-compression tests [1] by laboratory D

Mixture	Coarse aggregate	Fine aggregate	Filler	Air voids	Dry strength	Immersed 4 days at 120° F.		Immersed 4 days at 140° F.		Immersed 14 days at 120° F.	
						Wet Strength	Retained strength	Wet Strength	Retained strength	Wet Strength	Retained strength
				Percent	*P.s.i.*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
1	Traprock (Mass.)	Glacial sand (Mass.)	Traprock E (N. J.)	9.1	286	228	80			223	78
2	do	do	Traprock F (N. J.)	9.9	261	229	88			237	91
3	do	do	Rhyolite (Mass.)	8.7	293	70	24	48	16	42	14
4	do	do	Traprock (Mass.)	9.4	281	260	93	225	80		
5	do	do	Fly ash A (N. J.)	7.4	253	240	95	233	92	256	100+
6	do	do	Fly ash B (N. J.)	7.7	256	246	96	228	89	240	94
7	do	do	Fly ash C (N. J.)	7.4	255	242	95			218	85
8	do	do	Fly ash (Ill.)	7.3	263	252	96	251	95	228	87
9	Rhyolite (Mass.)	do	Traprock E (N. J.)	5.5	286	238	83	231	81		
10	do	do	Traprock F (N. J.)	6.0	245	252	100+	255	100+		
11	do	do	Rhyolite (Mass.)	4.4	294	130	44	50	17		
12	do	do	Traprock (Mass.)	5.1	272	265	97	238	88		
13	do	do	Fly ash A (N. J.)	3.7	254	248	98	244	96		
14	do	do	Fly ash B (N. J.)	3.4	273	233	85	241	88		
15	do	do	Fly ash C (N. J.)	3.5	267	244	91	243	91		
16	do	do	Fly ash (Ill.)	3.5	261	259	99	261	100		
17	do	Potomac River sand	do	4.5	259	282	100+	288	100+	257	99
18	do	do	Rhyolite (Mass.)	5.5	288	116	40	43	15	50	17
19	do	do	Mica dust (Md.)	6.9	278	253	91	215	77	225	81
20	Slag (Md.)	do	Rhyolite (Mass.)	6.5	365	277	76	103	28	90	25
21	do	do	Fly ash (Ill.)	5.4	342	312	91	305	89	287	84
22	do	do	Mica dust (Md.)	7.1	375	314	84	279	74	252	67

[1] All mixtures except Nos. 20, 21, and 22 contained 5.5 parts 85–100 penetration asphalt. In mixtures Nos. 20, 21, and 22 about two parts per 100 of additional asphalt were used to satisfy the absorption of the slag.

Table 17.—Summary of all mixtures of series 3

Filler	Coarse aggregate	Laboratory	Voids	Strength[1]		
				Dry	Wet	Retained
			Percent	*P.s.i.*	*P.s.i.*	*Percent*
Traprock dust (Mass.)	Traprock (Mass.)	B	6.8	301	258	86
do.[2]	do	B	6.1	324	275	85
do	do	A	7.7	433	393	[3] 91
do	do	A	8.5	560	514	[3,4] 92
do	do	D	9.4	281	260	93
do	Rhyolite (Mass.)	A	2.9	427	413	[3] 97
do	do	D	5.1	272	265	97
Traprock dust E (N. J.)	Traprock E (N. J.)	B	6.5	293	276	94
do	do	C	5.8	290	223	77
do	Traprock F (N. J.)	C	4.8	329	283	86
do	Traprock (Mass.)	E	9.5	339	263	78
do	do	D	9.1	286	228	80
do	Rhyolite (Mass.)	D	5.5	286	238	83
Traprock dust F (N. J.)	Traprock E (N. J.)	C	6.0	304	236	78
do	Traprock F (N. J.)	C	5.6	313	275	88
do	Traprock (Mass.)	E	9.9	355	271	76
do	do	D	9.9	261	229	88
do	Rhyolite (Mass.)	D	6.0	245	252	100+
Traprock dust (Va.)	Traprock (Va.)	B	4.2	363	339	93
Average percentage of retained strength, traprock dust						87
Limestone dust	Traprock (Mass.)	B	7.0	310	281	91
do.[2]	do	B	6.3	305	280	92
do	do	A	7.0	447	399	[3] 89
do	do	A	7.7	600	544	[3,4] 91
do	Traprock E (N. J.)	B	6.6	311	279	90
do	do	C	5.5	297	235	79
do	Traprock F (N. J.)	C	5.0	360	277	77
do	Traprock (Va.)	B	5.2	330	294	89
do	Rhyolite (Mass.)	B	4.6	304	271	89
do	do	A	2.9	443	406	[3] 92
do	Quartzite (Pa.)	B	4.4	352	314	89
Average percentage of retained strength, limestone dust						88
Fly ash A (N. J.)	Traprock E (N. J.)	C	4.3	235	226	96
do	Traprock F (N. J.)	C	4.0	291	277	95
do	Traprock (Mass.)	E	7.2	297	278	94
do	do	D	7.4	253	240	95
do	Rhyolite (Mass.)	D	3.7	254	248	98
Fly ash B (N. J.)	Traprock E (N. J.)	C	3.5	262	230	88
do	Traprock F (N. J.)	C	3.5	257	286	100+
do	Traprock (Mass.)	E	7.4	315	271	86
do	do	D	7.7	256	246	96
do	Rhyolite (Mass.)	D	3.4	273	233	85
Fly ash C (N. J.)	Traprock E (N. J.)	C	3.7	243	246	100+
do	Traprock F (N. J.)	C	3.6	334	285	85
do	Traprock (Mass.)	E	7.5	316	273	86
do	do	D	7.4	255	242	95
do	Rhyolite (Mass.)	D	3.5	267	244	91
Fly ash D (N. J.)	Traprock E (N. J.)	C	3.2	244	222	91
do	Traprock F (N. J.)	C	3.2	276	278	100+
Fly ash (Ill.)	Traprock (Mass.)	D	7.3	263	252	96
do	Rhyolite (Mass.)	D	3.5	261	259	99
do	do	D	4.5	259	282	100+
do	Slag (Md.)	D	5.4	342	312	91
Average percentage of retained strength, fly ash						94
Rhyolite dust (Mass.)	Traprock (Mass.)	A	7.0	428	376	[3] 88
do	do	A	8.5	630	533	[3,4] 85
do	do	D	8.7	293	70	24
do	Rhyolite (Mass.)	D	4.4	294	130	44
do	do	D	5.5	288	116	40
do	do	B	4.3	309	163	53
do	do	A	4.1	427	397	[3] 93
do	Slag (Md.)	D	6.5	365	277	76
Average percentage of retained strength, rhyolite dust						63
Quartzite dust (Pa.)	Quartzite (Pa.)	B	4.1	375	363	97
Silica dust (N. J.)	Traprock E (N. J.)	C	5.3	277	225	81
do	Traprock F (N. J.)	C	5.3	334	222	66
Mica dust (Md.)	Rhyolite (Mass.)	D	6.9	278	253	91
do	Slag (Md.)	D	7.1	375	314	84

[1] Retained strengths shown for 4-day, 120° F. immersion only.

[2] Sieved to pass No. 200 sieve.

[3] Molded specimens for the dry-strength test were cured 1 day at 140° F. and 14 additional days at 120° F. Specimens for wet-strength test cured 1 day at 140° F. and 10 additional days at 120° F. before immersion.

[4] Check test of mixture reported on line next above but no explanation available for increases in dry and wet strengths.

cept for the coarse aggregate. Both coarse aggregates are traprock but they are from different sources in New Jersey. The sand used throughout was the same Massachusetts glacial material as that used in the first and second series. The traprock-dust, silica-dust, and fly-ash fillers were all New Jersey materials.

All of the mixtures containing coarse aggregate E (upper half of table 12) would be considered satisfactory on the basis of the 4-day immersion test if, as has been suggested in connection with other studies, 75-percent retention is taken as the critical value. Of the mixtures containing coarse aggregate F, all would be considered satisfactory on the basis of the 4-day test except the one containing silica-dust filler.

Increasing the immersion period caused drastic additional loss of strength in both of the mixtures containing silica-dust filler but did not seriously affect the other 14 mixtures.

For the standard 4-day immersion, the percentage of original dry strength retained after immersion by the four traprock-dust mixtures ranged from 77 to 88, averaging 82. The retention for the eight fly-ash mixtures ranged from 85 to 100 percent, averaging 94. The two control mixtures, containing limestone dust, retained 77 and 79 percent of their dry strength after immersion. Thus, the traprock-dust mixtures were indicated by the test to be equal or superior to the limestone-dust mixtures and the fly-ash mixtures were indicated to be considerably superior to the limestone-dust mixtures.

The results of the cooperative tests by laboratory E are shown in table 13. Here the comparison is limited to traprock dust versus fly ash, with a single coarse aggregate, but the retention values are of the same order as those previously discussed—the traprock-dust mixtures being satisfactory according to the 75-percent criterion and the fly-ash mixtures rating considerably above the suggested critical value.

In tables 14 and 15, the results of the tests by laboratory A are shown. The work done by laboratory A in series 3 was of an exploratory nature and the proportions used, 6 parts of bitumen to 100 parts of aggregate, as well as various procedures that were followed, had no counterpart in the work of the other laboratories. Within themselves, however, the results of the tests by laboratory A show the same general trends as those discussed in connection with tables 11, 12, and 13. In the 4-day immersion test, as shown in table 15, the strength retention of the limestone-dust and traprock-dust mixtures was about the same, the minor differences being slightly in favor of the traprock dust. Tests X, Y, and Z were check runs against A, B, and C, respectively. The large differences between the dry and wet compressive strengths of the two sets of tests cannot be explained. In spite of these differences, the values for percentage of retained strength are in close agreement in the two sets of tests and, moreover, the relative values for traprock dust and limestone dust are in agreement with those of the other cooperating laboratories.

The results of the series 3 tests by laboratory D are shown in table 16. A direct comparison between traprock-dust and limestone-dust mixtures with 5½ parts of asphalt having been made in its series 2 tests, laboratory D concentrated mainly, in series 3, on obtaining a broad comparison

Table 18.—Comparison of traprock dust, fly ash, and limestone dust in series 2 and 3

Coarse aggregate	Reporting laboratory	Retained strength after immersion, 4 days, 120° F.									
		Traprock dust				Fly ash					Limestone dust
		Massachusetts	New Jersey		Virginia	New Jersey				Illinois	
			E	F		A	B	C	D		
Series 3 Mixtures (5.5 parts asphalt except as noted)											
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Traprock	B	86	94		93						91
do	B	[1] 85									[1] 92
do	B										90
do	B										89
do	C		77	78		96	88	100+	91		79
do	C		86	88		95	100+	85	100		77
do	E		78	76		94	86	86			
do	A [2]	91									89
do	A [2]	92									91
do	D	93	80	88		95	96	95		96	
Quartzite	B										89
Rhyolite	B										89
do	A [2]	97									92
do	D	97	83	100+		98	85	91		99	
do	D									100+	
Slag	D [3]									91	
Series 2 Mixtures (5.5 parts asphalt only)											
Traprock	B	80				93					80
do	D	93				100					88
Averages, series 3 and 2		90	83	86	93	96	91	91	96	97	87

[1] Filler sieved to pass No. 200 sieve. All others tested as received.

[2] Mixtures contained 6 parts asphalt to 100 parts of aggregate (basic). Curing non-standard (see table 15).

[3] Mixture contained 2 parts of extra asphalt to satisfy absorption of slag, or a total of 7.5 parts per 100 parts of aggregate, but may be considered to contain 5.5 parts of effective asphalt.

between traprock dust and fly ash. Miscellaneous fillers, an extra coarse aggregate, and an additional sand were also included in the program for the purpose of broadening the basis for conclusions.

In the standard 4-day immersion test, the traprock-dust mixtures consistently retained 80 percent, or more, of the dry strength. The values for the six mixtures ranged from 80 to 100 percent and averaged 90 percent. Thus, all of them would be rated as satisfactory on the basis of the 75-percent criterion. Among the ten mixtures of series 3 containing fly ash, only one retained less than 90 percent of its dry strength in the 4-day, 120° F. immersion test. The range of retentions for these ten mixtures was from 85 to 100 percent and the average was 95 percent. Thus, as in the other cooperating laboratories, the fly-ash mixtures were indicated to be superior to the traprock-dust mixtures in resistance to moisture.

The rhyolite dust did not show up well in the series 3 tests by laboratory D, especially with the crushed stone aggregates. None of the rhyolite mixtures with crushed stone would be considered satisfactory on the basis of the 4-day immersion test, and the one rhyolite-dust mixture with crushed slag, while barely exceeding the suggested 75-percent criterion, was at best a borderline case.

The special tests with the immersion temperature raised to 140° F. in one case, and the immersion time increased to 14 days in the other, did not add much information concerning the relative behavior of the various fillers to that shown in the standard test, immersion for 4 days at 120° F. In general, as compared to the standard test, these special tests slightly reduced the retained strength of mixtures containing traprock dust and fly ash, but not by significant amounts. They resulted in major reductions in retained strength of the mixtures containing rhyolite filler which the standard test had already indicated to be unsatisfactory.

The tests on the natural mica dust from Maryland showed it to be about equivalent in its laboratory test behavior to the traprock dusts.

The change from glacial sand to Potomac River sand in some of the mixtures containing rhyolite coarse aggregate (see table 16) caused no significant change in the test values. Compare mixture No. 16 with No. 17, and mixture No. 11 with No. 18. Therefore, in comparing the test results on the various mixtures, the type of sand used in these tests need not be regarded as a variable factor.

Summary

All the data from tables 11–16 that were obtained from the 4-day, 120° F., immersion test have been assembled in table 17 in order to present an over-all picture for series 3. The average percentages of retained strength for the mixtures containing the various fillers indicate rhyolite to be unsatisfactory, limestone dust and traprock dust essentially equal to each other and well within the class of satisfactory materials, and fly ash superior to all the rest.

In table 18 the data from table 17, series 3 tests, pertaining to traprock dust, fly ash, and limestone dust, and those from table 6, series 2 tests, that contained 5½ parts of asphalt, have been arranged so that the range of values for retained strength obtained by the various laboratories for each of the filler samples can be more readily seen. Table 18 also indicates the degree of agreement among the cooperating laboratories on test values for individual material combinations. It is especially interesting to note that, despite some lack of agreement on the dry and wet compressive strength values and percentages of retained strength, the individual laboratories all show the same relative performance of these three filler materials. This agreement has been apparent in the preceding discussion of the individual laboratory reports, tables 11–16. Thus, the main objective of the study, to evaluate the relative laboratory performance of traprock dust, fly ash, and limestone dust as fillers in bituminous concrete, appears to have been achieved.

The related studies that were conducted by most of the cooperators, to explore the need for higher immersion temperatures or longer immersion periods in the exposure test, were of definite value in two ways. First, they indicated that generally the 4-day test at 120° F. can be relied upon to give a satisfactory separation between good and poor fillers, and second, they showed that usually, for fillers of bad repute, the extension of the 120° F. immersion to 14 days greatly magnified the detrimental effect of the water whereas, in the case of the resistant fillers, the extended exposure caused little additional damage over that resulting from the 4-day immersion. Thus, if for any reason it should be felt that a given material showing satisfactory behavior in the 4-day test might actually not be a satisfactory filler in practice, it could be checked at the longer immersion period to clear up any doubt.

There appears to be no evidence that 4-day immersion at 140° F. would provide any more convincing information than the standard exposure for 4 days at 120° F.

Some of the other supplemental studies conducted by individual laboratories have also been useful and informative. Those of laboratory B, mentioned in connection with the discussion of table 7, showed a slight tendency for the fine portion of the mineral powder to act more effectively as a filler than when the portion retained on the No. 200 sieve was included. Those of laboratories B and C, discussed in connection with tables 8 and 9, showed in a striking way the misleading strength values that may be expected when cooled mixtures are reheated for molding or when previously compacted mixtures are broken up, reheated, and molded.

Laboratory D made chemical analyses of four fly-ash samples, three traprock-dust samples, and one sample of rhyolite dust. The results of these analyses are given in table 19. They are presented to make available as much information as possible concerning the materials used but they are not sufficiently extensive to permit a study of the possible effect of chemical composition on the behavior of the fillers. It may be noted that the fly-ash samples showed the lowest silica content (41 to 48 percent) and the best behavior in the test mixtures, while the traprock samples with slightly higher silica contents (50 to 52 percent) rated second in performance. The rhyolite, with the highest silica content (64 percent), showed generally unsatisfactory performance in the test mixtures.

Table 19.—Chemical analyses of fly ash, traprock, and rhyolite fillers

	Illinois	New Jersey—					Massachusetts—	
	Fly ash	Fly ash A	Fly ash B	Fly ash C	Traprock E	Traprock F	Traprock	Rhyolite
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Ignition loss (1,000° C.) [1]	3.0	4.1	2.4	5.0	2.3	2.0	3.4	3.1
Chemical analysis:								
SiO_2	45.3	47.8	41.2	42.8	50.9	52.1	49.9	64.1
Al_2O_3	20.2	14.7	24.1	30.0	14.6	17.0	14.5	14.9
Fe_2O_3	19.4	28.6	22.1	16.1	11.3	12.0	14.0	5.3
CaO	6.3	1.5	5.6	3.7	9.7	8.6	9.3	3.6
MgO	1.1	1.2	1.3	.7	7.9	4.7	6.1	2.0
SO_3	1.8	.5	1.4	.6	.2	.1	....	....
Na_2O	1.3	.3	.7	.3	2.6	2.2	2.6	3.8
K_2O	2.0	1.9	1 2	1.4	.6	.8	.7	3.1
Total	100.4	100.6	100.0	100.6	100.1	99.5	100.5	99.9

[1] Any free carbon present is included in this percentage. In the case of the fly-ash samples, it is likely that a major part of the ignition loss represents free carbon.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1952

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$3,432	$32,212	$16,361	518.3	$5,302	$2,706	103.5	$25,582	$13,075	411.7	$63,096	$32,142	1,033.5
Arizona	802	2,327	1,495	60.2	772	375	13.9	8,251	5,038	106.7	11,350	6,908	180.8
Arkansas	2,976	9,432	5,043	272.7	2,334	1,184	81.1	15,367	7,851	382.4	27,133	14,078	736.2
California	1,222	5,294	2,865	72.5	9,349	5,131	61.9	93,791	44,661	229.3	108,434	52,657	363.7
Colorado	1,574	6,605	3,699	134.2	1,696	943	53.1	9,952	4,884	140.1	18,253	9,526	327.4
Connecticut	4,159	2,280	1,242	7.8	4,063	2,026	10.3	9,379	4,765	15.7	15,722	8,033	33.8
Delaware	1,094	964	482	2.8	1,091	546	4.8	6,471	3,273	33.5	8,526	4,301	41.1
Florida	843	14,262	7,301	186.2	12,009	6,030	204.8	12,960	6,668	166.0	39,231	19,999	557.0
Georgia	2,120	15,867	8,171	356.5	7,119	3,639	99.1	36,181	17,167	545.5	59,167	28,977	1,001.1
Idaho	3,262	8,644	5,167	254.1	2,075	1,336	45.0	6,956	4,438	98.6	17,675	10,941	397.7
Illinois	6,886	36,084	19,565	358.9	18,469	10,378	107.4	71,205	35,836	634.8	125,758	65,779	1,101.1
Indiana	7,110	32,851	16,816	149.8	6,826	3,578	118.4	27,349	14,565	227.3	67,026	34,959	495.5
Iowa	2,102	12,260	6,630	283.2	964	516	49.7	20,894	10,477	958.1	34,118	17,623	1,291.0
Kansas	3,862	8,361	4,102	930.2	5,806	2,820	469.3	17,722	9,246	734.9	31,889	16,168	2,134.4
Kentucky	827	15,549	8,535	242.2	6,475	3,315	148.3	16,887	8,375	262.9	38,911	20,225	653.4
Louisiana	1,830	12,796	6,196	93.4	5,671	2,809	35.0	24,418	11,725	168.8	42,885	20,730	297.2
Maine	152	6,892	3,691	30.4	894	491	8.6	12,788	6,269	97.4	20,574	10,451	136.4
Maryland	4,913	7,836	3,605	58.0	2,916	1,294	31.1	9,999	5,375	30.3	20,751	10,274	119.4
Massachusetts	3,116	4,194	2,242	18.6	867	436		47,652	22,771	47.0	52,713	25,449	65.6
Michigan	1,208	21,731	11,031	349.3	4,567	2,294	103.3	61,990	27,097	321.8	88,288	40,422	774.4
Minnesota	2,784	8,350	4,704	880.1	1,354	790	127.3	19,257	10,364	648.2	28,961	15,858	1,655.6
Mississippi	1,866	14,486	7,349	533.6	4,902	2,569	154.9	16,410	8,542	512.5	35,798	18,460	1,201.0
Missouri	6,719	24,736	12,602	855.1	8,608	4,307	128.5	37,567	19,690	484.0	70,911	36,599	1,467.6
Montana	5,898	8,692	5,021	264.4	1,331	793	12.5	14,170	8,502	246.4	24,193	14,316	523.3
Nebraska	11,768	9,974	5,350	424.2	634	373	29.6	16,104	7,914	487.3	26,712	13,637	941.1
Nevada	1,868	5,711	4,480	185.2	1,673	1,097	91.3	2,733	2,236	70.3	10,117	7,813	346.8
New Hampshire	1,350	3,878	1,939	21.0	763	380	2.5	3,822	2,043	22.5	8,463	4,362	46.0
New Jersey	1,545	8,593	4,240	36.5	6,957	3,451	3.6	32,735	16,201	36.1	48,285	23,892	76.2
New Mexico	798	2,027	1,297	64.4	3,041	1,949	111.6	7,545	4,824	212.6	12,613	8,070	388.6
New York	11,695	83,222	43,916	151.2	38,969	19,109	60.6	127,711	57,923	498.6	249,902	120,948	710.4
North Carolina	2,160	20,145	9,747	326.4	5,301	2,518	91.5	27,620	13,473	496.7	53,066	25,738	914.6
North Dakota	2,084	4,383	2,324	707.6	1,592	794	287.3	9,346	4,713	812.9	15,321	7,831	1,807.8
Ohio	6,864	16,189	7,980	109.0	6,214	3,634	21.6	96,843	47,501	195.5	119,246	59,115	326.1
Oklahoma	2,925	10,540	5,878	158.8	7,700	3,886	163.3	21,333	11,316	244.8	39,573	21,080	566.9
Oregon	357	1,891	1,040	2.4	1,961	1,212	16.1	14,538	8,189	214.1	18,390	10,441	232.6
Pennsylvania	6,509	21,401	10,693	45.4	11,428	5,706	62.7	86,654	43,091	187.7	119,483	59,490	295.8
Rhode Island	836	3,033	1,516	27.7	920	460	2.7	17,745	9,249	26.7	21,698	11,225	57.1
South Carolina	1,609	11,139	6,024	198.7	2,293	1,184	141.9	15,710	8,003	308.9	29,142	15,211	649.5
South Dakota	582	5,689	3,244	469.0	2,097	1,357	120.0	7,994	4,726	456.7	15,780	9,327	1,045.7
Tennessee	2,528	9,779	4,863	361.3	8,019	4,017	113.4	31,446	14,466	392.7	49,244	23,346	867.4
Texas	5,473	3,545	1,644	118.5	12,103	6,469	369.1	55,340	29,533	992.5	70,988	37,646	1,480.1
Utah	890	3,683	2,839	115.0	1,030	776	19.8	9,251	6,987	148.8	13,964	10,602	283.6
Vermont	1,040	3,353	1,861	30.5	540	270	.8	6,755	3,396	56.1	10,648	5,527	87.4
Virginia	2,252	11,309	5,647	191.3	8,227	4,025	102.1	25,981	12,530	267.8	45,517	22,202	561.2
Washington	1,801	7,849	4,269	112.5	2,993	1,697	53.3	12,915	6,742	114.3	23,757	12,708	280.1
West Virginia	1,452	7,495	3,821	44.5	5,135	2,595	59.2	17,590	8,770	134.5	30,220	15,186	238.2
Wisconsin	3,519	11,316	6,615	184.5	2,409	1,001	65.1	34,644	16,976	403.5	48,369	24,592	653.1
Wyoming	215	709	483	18.7	998	672	30.5	8,398	5,541	149.8	10,105	6,696	199.0
Hawaii	928	3,204	1,579	6.4	542	265	15.1	12,983	5,827	31.3	16,729	7,671	52.8
District of Columbia	1,048	13,156	5,757	1.2	70	35	.2	4,685	2,339	.5	17,911	8,131	1.9
Puerto Rico	3,117	7,535	3,689	51.9	2,254	1,044	10.6	12,870	6,165	45.8	22,659	10,898	108.3
TOTAL	147,970	603,453	316,650	11,076.3	251,323	130,282	4,217.3	1,344,489	671,328	14,512.9	2,199,265	1,118,260	29,806.5

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VOL. 27, NO. 6

FEBRUARY 1953

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Late afternoon traffic on the Orleans Street Viaduct, Baltimore, Md.

Public Roads

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

Trends in Traffic Volumes, Vehicle Types and Weights

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
THOMAS B. DIMMICK,
Head, Current Data Analysis Unit

Total travel on all rural roads in 1951 broke all records, exceeding the 1950 previous high by 10 percent. On the 356,000 miles of main rural roads in the United States, travel in 1951 was almost 190 billion vehicle-miles, of which 78 percent was by passenger cars, 1 percent by busses, and 21 percent by freight-carrying vehicles.

Trucks and combinations hauled 4 percent more ton-mileage of freight on main rural roads in 1951 than in 1950. Single-unit truck travel was 2 percent higher than in 1950 while that of combinations increased about 1 percent. The average carried load for all trucks and combinations in 1951 was less than 1 percent above the average in 1950.

In 1951, 5 percent of all trucks and combinations exceeded a State legal weight limit, and 14 percent of the combinations were illegally overloaded in some particular. In comparison with 1950, the percentage of overweight vehicles for 1951 decreased in all regions except New England and the West North Central States.

RURAL MOTOR-VEHICLE TRAVEL broke all previous records in 1951 for the sixth consecutive year. The estimated 1951 traffic on all rural roads was over 10 percent above the 1950 total, 20 percent higher than in 1949, almost 31 percent higher than in 1948, slightly more than 39 percent higher than in 1947, somewhat more than 52 percent higher than in 1946, and almost 53 percent higher than in 1941. Data collected from January through August indicate that travel on all rural roads in 1952 will continue the same general trend and will be almost 7 percent higher than in 1951.

The variation in average daily travel on rural roads by months in the three main geographical divisions[1] and in the United

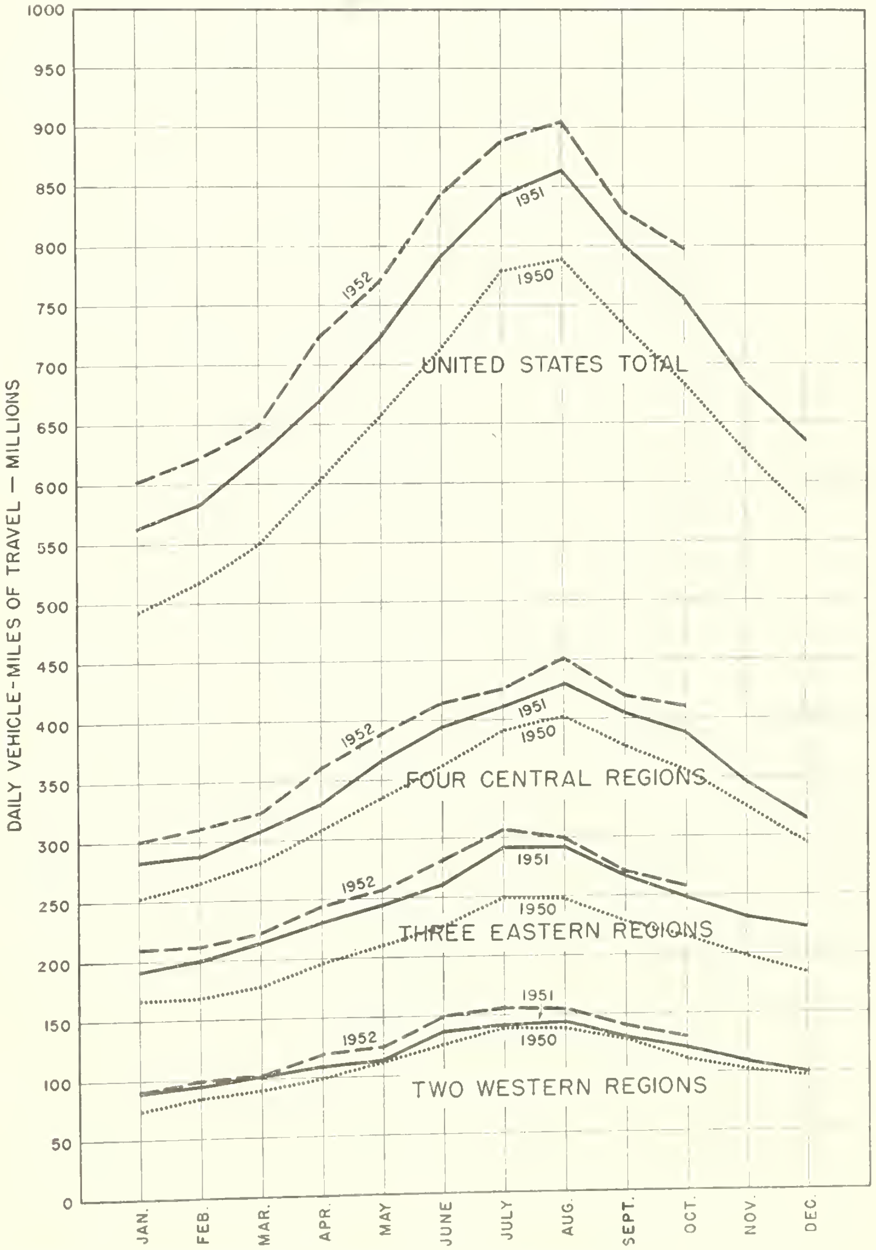

Figure 1.—Travel on all rural roads in 1950, 1951, and in the first 10 months of 1952.

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[1] The States comprising each census region, and the regions comprising each geographic division, are indicated in table 1.

States as a whole is illustrated in figure 1 for the years 1950, 1951, and the first 10 months of 1952. Travel in each month of these years in the Eastern and Central regions and in the United States as a whole was well above that of the corresponding month of the earlier year. The Western region showed only slight gains in January and March of 1952 compared to the amount of travel in 1951 but a fairly steady gain in all other months.

For the three Eastern regions, the chart indicates a somewhat greater increase than that shown by continuous counts at fixed locations because of the upward adjustments of the vehicle-mileage following recent surveys and analyses by Georgia and New York, more comprehensive than those previously made. Similar adjustments in other regions, following new surveys and analyses, did not result in changes of appreciable significance.

Approximately the same rate of increase in 1952 over 1951 is indicated by data collected in the first portion of the later year. The partial 1952 information indicated increases of 6 percent in the Central States and 7 percent in the Eastern and Western States. The largest indicated increase over 1951 in any census region was 10 percent in the East South Central; the smallest increase was 5 percent in the Middle Atlantic region.

Summer travel constituted a smaller portion of the annual travel in 1951 than in any recent year. In the last two prewar years (1940 and 1941), the average daily traffic in July and August was 23 percent above the average traffic for the year. Not until 1949 did the summer travel reach the prewar ratio. In 1950 the average daily summer travel was over 24 percent above the annual average daily amount, but this figure slumped to 22 percent in the 1951 summer season.

Figure 2, showing travel on all rural roads by 12-month periods ending each month (moving average) and as a percentage of traffic in the calendar year 1941, gives an accurate picture of the effect of wartime restrictions on prewar travel and the steady growth of traffic that has occurred since the end of hostilities. The increase in traffic from the end of 1946 to the present has averaged almost 9 percent, compounded annually. From these data it is apparent that the general pattern of traffic growth is being maintained with no evidence of its leveling off.

The lower portion of figure 2, showing the relation of travel by 12-month periods in each of the main geographical regions of the United States to that in the calendar year 1941, shows clearly how much travel was shifted westward during the war. The spread between the curves for the three regions remained constant during 1947 and 1948, was reduced slightly during 1949 and 1950, and was greatly reduced in 1951 and the portion of 1952 shown.

Basis of Estimates

During certain prewar years, generally 1936 or 1937, nearly every State conducted a comprehensive survey of traffic in which all vehicles counted were classified by type. At the same time a large number of trucks and truck combinations were stopped and weighed and information recorded concerning their weight, dimensions, and other important features.

While the large number of automatic recorders operated on the rural roads of

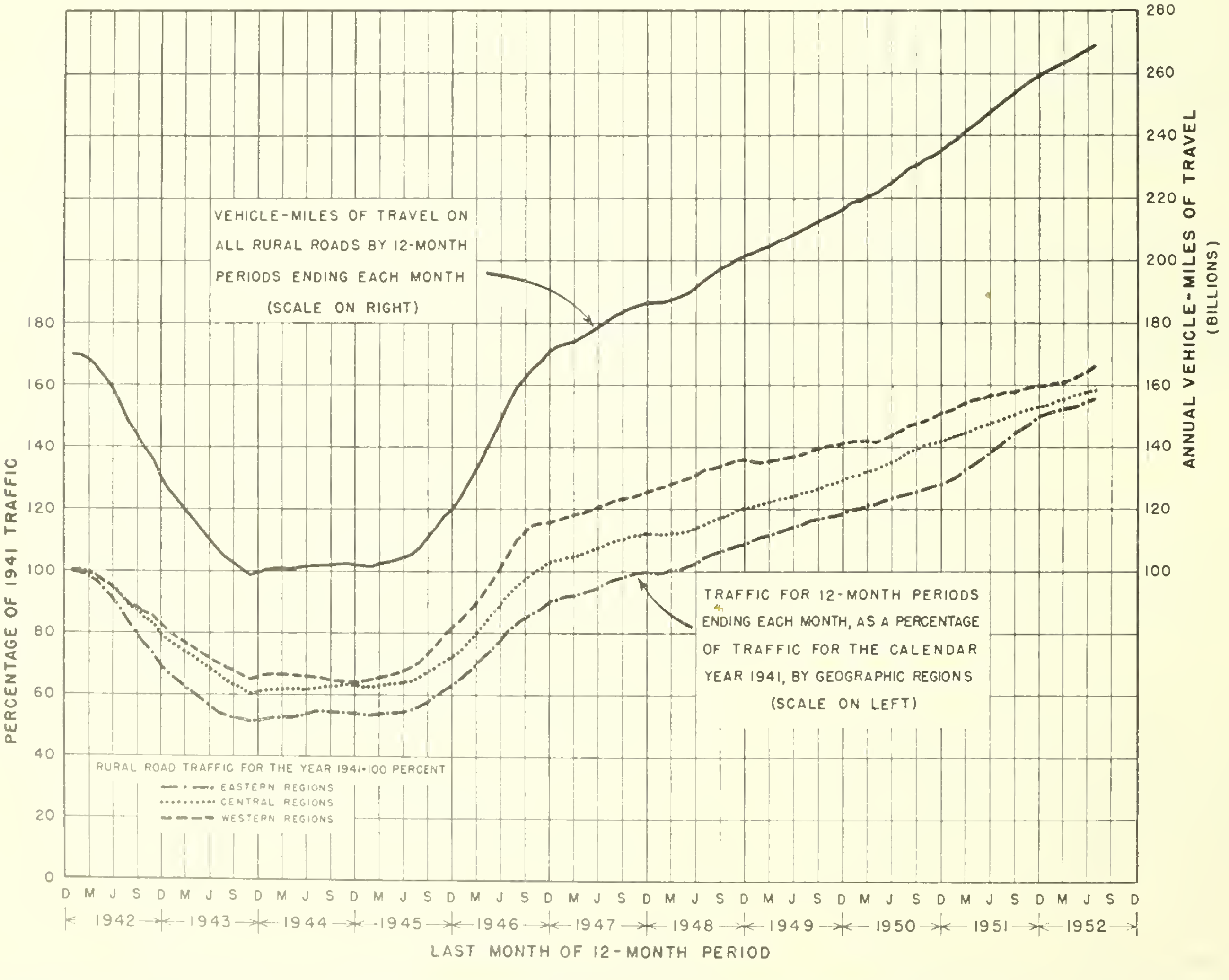

Figure 2.—Travel on all rural roads by 12-month periods ending each month, in vehicle-miles and as a percentage of traffic in the calendar year 1941.

each State give a good indication of the trend of total traffic on these highways, they provide no indication of the classification of vehicles by type, weight, or other characteristics. In order to determine the trends in such factors, brief check surveys were made in the summer of 1942 at certain typical stations in most States. From strictly comparable information gathered in the two surveys, trends were calculated which were used to determine the changes in traffic and vehicle characteristics that had taken place since the comprehensive survey was made. Since 1942, check surveys have been made annually. Most States have participated in these each year and all have participated at some time.[2] Such surveys were conducted in 45 States in 1951, although two States, Virginia and Washington, were unable to analyze their data in time to be included in this report.

Classification counts made in numerous States, in addition to those made at weight stations, added valuable information concerning vehicle-type proportions. Greatly expanded loadometer surveys made throughout the year in a few States have furnished more reliable data concerning vehicle types and weights than can be obtained from trend data alone. Starting in the summer of 1950, a number of weight stations were selected in 26 States from which data concerning weights and characteristics of truck traffic were to be used in studying loads and their relation to pavement damage. These stations generally are operated 16 or 24 hours a day and at least one day each season. The new data derived from these more extensive operations give more information concerning traffic characteristics in various hours of the day and seasons of the year than has been available at any time since the time of the initial surveys. All of these data wherever available have been used in the estimates.

1951 Summer Loadometer Survey

The stations used in the check surveys were selected initially to give a representative cross section of traffic on main rural roads. They were operated for one or more 8-hour periods on a weekday, generally from either 6 a.m. to 2 p.m. or from 2 p.m. to 10 p.m. All traffic passing through the stations during the period was counted and classified into the following categories: local passenger cars; foreign (out-of-State) passenger cars; panel and pick-up trucks; other two-axle, four-tire trucks; two-axle, six-tire trucks; three-axle trucks; truck-tractor and semitrailer combinations; truck and trailer combination or truck-tractor semitrailer and trailer combinations; and busses. The combination-type vehicles were further subdivided according to number of axles of each.[3]

Most of the weight stations were operated during July, August, and September. The survey period, number of stations operated, number of vehicles counted, and the number weighed are shown for each State in table 1. More than 1.7 million vehicles were

Table 1.—Survey period, number of stations operated, number of vehicles counted, and number weighed in each State in the special weight surveys, summer of 1951

Region and State	Survey period	Number of stations	Total vehicles counted	Trucks and truck combinations	
				Counted	Weighed
New England:					
Connecticut	Aug. 1–Aug. 24	10	36,695	6,820	1,665
Maine	July 5–Aug. 3	9	46,899	8,132	3,334
Massachusetts	July 23–Aug. 13	10	116,163	14,817	4,380
New Hampshire	No survey	.....	.....	.....	.....
Rhode Island	July 16–Aug. 9	5	37,703	4,921	2,145
Vermont	July 23–Aug. 9	5	22,890	1,834	1,698
Subtotal	.....	39	260,350	36,524	13,222
Middle Atlantic:					
New Jersey	July 30–Aug. 8	10	96,500	15,205	2,069
New York	July 18–July 30	20	34,425	8,072	1,522
Pennsylvania	July 16–Aug. 31	13	70,602	15,425	3,133
Subtotal	.....	43	201 527	38,702	6,724
South Atlantic:					
Delaware	Aug. 9–Aug. 27	8	28,921	6,033	1,106
Florida	No survey	.....	.....	.....	.....
Georgia	Aug. 20–Sept. 25	15	37,116	9,663	4,489
Maryland	June 18–July 17	10	99,676	20,042	3,588
North Carolina	Aug. 2–Aug. 28	12	31,975	6,946	4,153
South Carolina	Sept. 17–Sept. 28	10	19,783	5,276	2,095
Virginia	Survey not reported	.....	.....	.....	.....
West Virginia	Aug. 14–Aug. 30	9	14,494	3,583	1,513
Subtotal	.....	64	231,965	51,543	16,944
Eastern regions, subtotal	.....	146	693,842	126,769	36,890
East North Central:					
Illinois	No survey	.....	.....	.....	.....
Indiana	Aug. 8–Sept. 6	20	55,770	13,171	4,724
Michigan	June 12–July 18	9	29,823	6,202	2,208
Ohio	July 10–July 26	10	29,570	5,694	1,514
Wisconsin	July 2–Sept. 25	19	90,241	13,410	3,181
Subtotal	.....	58	205,404	38,477	11,627
East South Central:					
Alabama	July 17–Aug. 21	10	32,599	8,121	6,182
Kentucky	July 11–Aug. 17	10	39,679	9,287	3,470
Mississippi	July 11–July 6	15	27,469	6,757	3,734
Tennessee	Aug. 21–Sept. 5	10	15,076	4,203	1,756
Subtotal	.....	45	114,823	28,368	15,142
West North Central:					
Iowa	July 1–Aug. 31	10	33,348	5,932	5,220
Kansas	Aug. 10–Sept. 13	10	10,484	2,260	571
Minnesota	June 4–Oct. 18	14	59,110	10,677	3,671
Missouri	July 30–Aug. 28	21	176,641	32,948	11,973
Nebraska	July 19–Aug. 22	20	27,015	5,604	5,373
North Dakota	July 10–Aug. 31	14	24,179	5,036	1,811
South Dakota	July 30–Sept. 17	15	24,174	3,899	3,601
Subtotal	.....	104	354,951	66,356	32,220
West South Central:					
Arkansas	July 23–Aug. 3	10	17,381	5,802	1,371
Louisiana	July 30–Aug. 13	10	13,089	3,620	1,138
Oklahoma	July 11–Aug. 10	10	28,995	6 526	5,983
Texas	June 1–Aug. 31	20	108,115	23,583	5,535
Subtotal	.....	50	167,580	39,531	14,027
Central regions, subtotal	.....	257	842,758	172,732	73,016
Mountain:					
Arizona	July 16–July 27	10	10,291	2,245	590
Colorado	July 2–Sept. 4	13	26,226	3,664	846
Idaho	Aug. 7–Sept. 6	10	15,441	3,003	1,484
Montana	July 30–Aug. 31	11	27,498	5,121	2,118
Nevada	July 31–Aug. 17	10	8,924	1,560	708
New Mexico	Aug. 6–Aug. 20	10	15,194	3,372	1,516
Utah	July 16–Aug. 3	10	19,879	3,672	1,092
Wyoming	July 31–Aug. 20	10	12,610	2,255	767
Subtotal	.....	84	136,063	24,892	9,121
Pacific:					
California	June 7–July 18	17	[1]72,180	13,902	4,226
Oregon	Aug. 8–Aug. 23	8	19,852	3,965	2,187
Washington	Survey not reported	.....	.....	.....	.....
Subtotal	.....	25	92,032	17,867	6,413
Western regions, subtotal	.....	109	228,095	42,759	15,534
United States total	.....	512	1,764,695	342,260	125,440

[1] Passenger cars not counted; figure given is an estimate based on data from other reports.

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[2] See *Traffic trends on rural roads*, by T. B. Dimmick, PUBLIC ROADS, vol. 26, No. 11, Dec. 1951; vol. 26, No. 5, Dec. 1950; vol. 25, No. 12, Feb. 1950; vol. 25, No. 7, Mar. 1949; vol. 25, No. 3, Mar. 1948; vol. 24, No. 10, Oct.-Nov.-Dec. 1946; and *Amount and characteristics of trucking on rural roads*, by J. T. Lynch and T. B. Dimmick, PUBLIC ROADS, vol. 23, No. 9, July-Aug.-Sept. 1943.

[3] In this article, the term "truck" is used to indicate a single-unit vehicle; "truck combination" to indicate truck-tractor semitrailer (with or without full trailer) and truck with full trailer; and "truck and truck combinations" or "trucks and combinations" to indicate all of these vehicles together.

counted at all stations during the period of the survey. Almost one-fifth of these were freight-carrying vehicles, of which almost 37 percent were weighed.

Wherever traffic volume permitted, each truck and truck combination was stopped and weighed. Where this procedure was impracticable, all of the less common types were weighed and the common vehicle types were weighed in sufficient numbers to establish their characteristics from the sample. The type of vehicle, whether loaded or empty, the number of axles, and the weight of each axle were recorded. The axle-spacing and the total wheelbase length of the heavier vehicles[4] were measured. Passenger cars and busses were counted but not stopped for weighing.

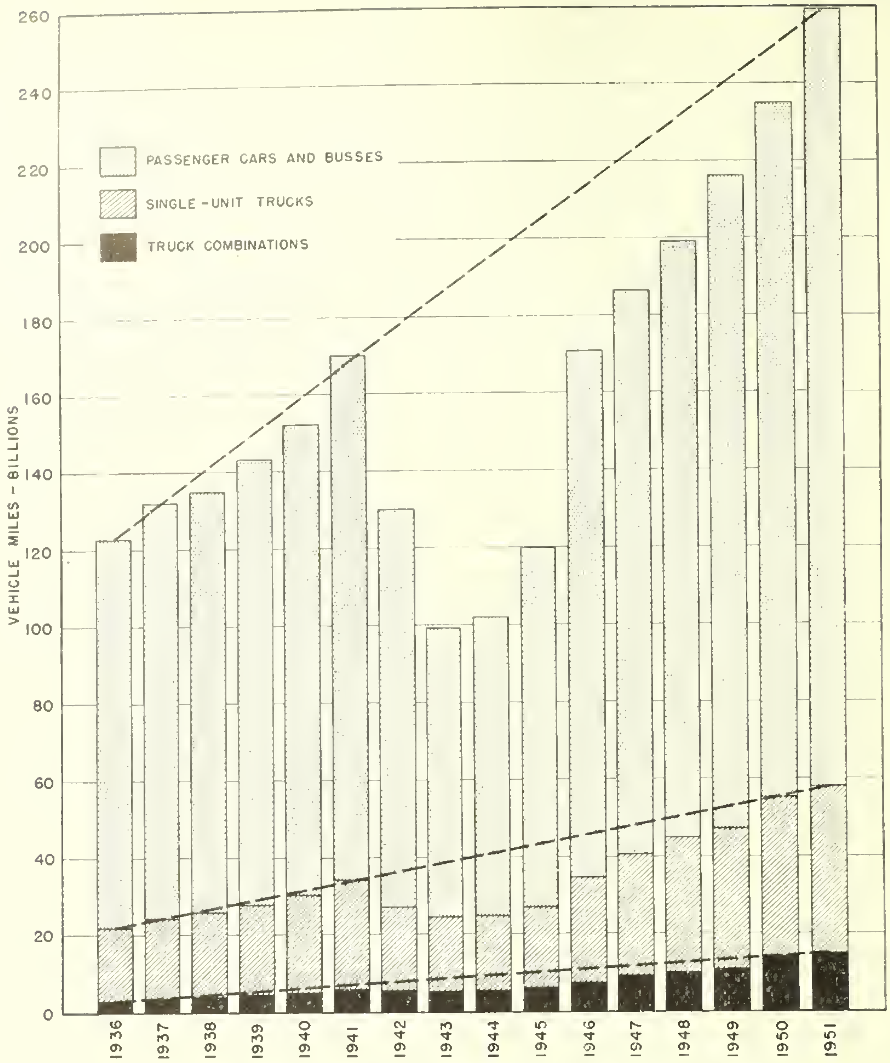

Figure 3.—Travel on all rural roads, 1936–51, by classes of vehicles.

Prewar Traffic Trend Increased

Figure 3 shows in chart form the vehicle-mileage of travel on all rural roads, by vehicle types, for each year from 1936 to 1951, inclusive. It is apparent that the effect of the drastic restrictions on travel during the war period, 1942-45, caused but a temporary dip in traffic growth and that the 1951 vehicle-mileage was as high as would have been estimated by any rational projection of the prewar trend. A straight line from the top of the bar for 1936 to the top of the bar for 1951 passes through the top of the bar for 1937 and for 1941 and falls well above the tops of the bars for all other years. This line indicates an average (simple) increase over the 15 years of almost 7.4 percent a year. This is equivalent to a rate of 5.1 percent compounded annually. The period 1936 to 1941 has much in common with the period 1946 to 1951, inclusive. During both periods the trend was quite uniform for the first 4 years but there was an upsurge of traffic in the fifth year. Traffic levels were considerably higher in the 1946 to 1951 period than in the earlier one, and the rate of increase was greater. During the earlier 5-year period the total increase was 38 percent, while during the later 5-year period it was 52 percent, or an average of 8.7 percent a year, compounded annually.

Travel by trucks and truck combinations increased in a manner very similar to that observed for all vehicles. For truck combinations alone, the 1936–51 line lies above the tops of all bars from 1937 to 1949, inclusive, thus showing an accelerating upward trend in the travel by these heavier vehicles. This trend is emphasized by other data given in other portions of this report.

As previously stated, the traffic data collected at automatic traffic-recorder stations in the first half of 1952 indicate that travel by all types of vehicles has continued to increase over that in the same months of

Table 2.—Ratio of 1951 traffic on main rural roads to corresponding traffic in 1950[1]

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	1.19	1.08	1.22	1.15	1.07	1.17	1.16	1.15	1.12	1.15	1.06	1.08	1.13
Foreign	.98	.92	1.18	1.05	1.05	1.17	1.07	1.07	1.08	.94	1.19	1.03	1.06
All passenger cars	1.13	1.04	1.21	1.13	1.06	1.17	1.14	1.13	1.11	1.06	1.07	1.07	1.11
Trucks and combinations													
Single-unit trucks	.83	1.01	1.04	1.00	1.00	.98	.97	1.08	1.01	.99	1.21	1.10	1.02
Truck combinations	.92	1.00	.94	.96	1.00	1.05	1.04	1.10	1.03	.92	1.10	1.04	1.01
All trucks and combinations	.85	1.01	1.01	.99	1.00	1.00	.99	1.09	1.02	.97	1.17	1.08	1.02
Busses	1.09	1.22	1.00	1.08	.96	.95	.95	1.00	.97	.87	1.00	.95	1.00
All vehicles	1.07	1.04	1.16	1.10	1.05	1.11	1.10	1.12	1.09	1.04	1.09	1.07	1.09

[1] The ratios for "all vehicles" are based on year-around automatic recorder data, while those for the individual vehicle types are based principally on summer counts.

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[4] Trucks and truck combinations weighing 13 tons or more, or having an axle weighing 18,000 pounds or more.

Table 3.—Percentage distribution of travel, by vehicle type, on main rural roads in the summer of 1951

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	61.47	64.32	59.57	61.66	55.71	46.27	62.19	60.29	57.01	48.10	68.80	61.21	59.26
Foreign	22.31	14.31	19.17	17.72	22.77	23.94	15.14	14.30	19.03	29.78	11.39	18.05	18.43
All passenger cars	83.78	78.63	78.74	79.38	78.48	70.21	77.33	74.59	76.04	77.88	80.10	79.29	77.69
Single-unit trucks:													
Panel and pick-up	4.12	4.81	6.83	5.68	5.30	10.70	7.21	11.10	7.98	9.57	5.04	6.69	7.00
Other 2-axle, 4-tire	.80	1.06	.52	.77	.29	.41	.61	.43	.42	.63	1.10	.93	.62
Other 2 axle, 6-tire	5.83	7.21	6.42	6.65	6.06	10.64	8.07	5.92	7.15	5.67	4.56	4.97	6.62
3-axle	.32	.50	.41	.43	.32	.35	.32	.16	.28	.42	1.10	.85	.43
All single-unit trucks	11.07	13.58	14.18	13.53	11.97	22.10	16.21	17.61	15.83	16.29	11.80	13.44	14.67
Truck-tractor and semitrailer combinations:													
3-axle	3.51	5.11	3.56	4.15	4.38	5.01	2.45	3.76	3.87	1.58	1.09	1.27	3.52
4-axle	0.39	1.70	2.43	1.87	3.56	1.22	2.58	2.86	2.82	1.18	1.06	1.10	2.22
5-axle or more	(1)	.02	.01	.01	.21	.02	.43	.06	.20	1.33	2.99	2.39	.50
All truck-tractor and semitrailer combinations	3.90	6.83	6.00	6.03	8.15	6.25	5.46	6.68	6.89	4.09	5.14	4.76	6.24
Truck and trailer combinations:													
4-axle or less	.01	.04	.01	.02	.15	.01	.27	.25	.18	.14	.33	.26	.14
5-axle	...	.01	...	.01	.43	...	.01	(1)	.16	.46	.88	.73	.21
6-axle or more		...	...	...	.09	...	.01	...	.04	.19	.84	.60	.12
All truck and trailer combinations	.01	.05	.01	.03	.67	.01	.29	.25	.38	.79	2.05	1.59	.47
All combinations	3.91	6.88	6.01	6.06	8.82	6.26	5.75	6.93	7.27	4.88	7.19	6.35	6.71
All trucks and truck combinations	14.98	20.46	20.19	19.59	20.79	28.36	21.96	24.54	23.10	21.17	18.99	19.79	21.38
Busses	1.24	.91	1.07	1.03	.73	1.43	.71	.87	.86	.95	.91	.92	.93
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

[1] Less than 0.005 percent.

1951. Available information indicates that travel on rural roads in 1952 will exceed the 1951 total by about 7 percent.

Travel Increases

The State system of highways in most States is composed of the main rural roads, or those on a connected system carrying the heaviest traffic. In such States as North Carolina, Pennsylvania, and Virginia, where all or a large part of the rural mileage is under State control, only the primary roads are included in this report as a part of the "main" system of highways of the country. These main roads, comprising about 356,000 miles, include less than 12 percent of the total rural mileage but carry over 73 percent of the total rural traffic. Because of the greater importance of these highways, from a traffic standpoint, most of the current traffic data were collected at points on them, and the remainder of this report will be concerned only with information concerning this portion of the road mileage.

The ratio of traffic volumes on main rural roads in 1951 to corresponding volumes in the previous year is shown in table 2. Although travel in 1951 on the main highways was higher in every region than in 1950, and the general increase for the United States as a whole was the same as in the previous year, the increase in travel of single-unit trucks, truck combinations, and foreign (out-of-State) passenger cars was considerably less than in the previous year and travel in these categories decreased in several regions. For instance, the table indicates that in 1951 travel by trucks was less in New England, East South Central, West North Central, and Mountain regions than in 1950 while truck combination traffic was less in New England, South Atlantic, and Mountain regions than in 1950. Likewise, travel by foreign passenger cars was less in the New England, Middle Atlantic, and Mountain regions while bus traffic in-

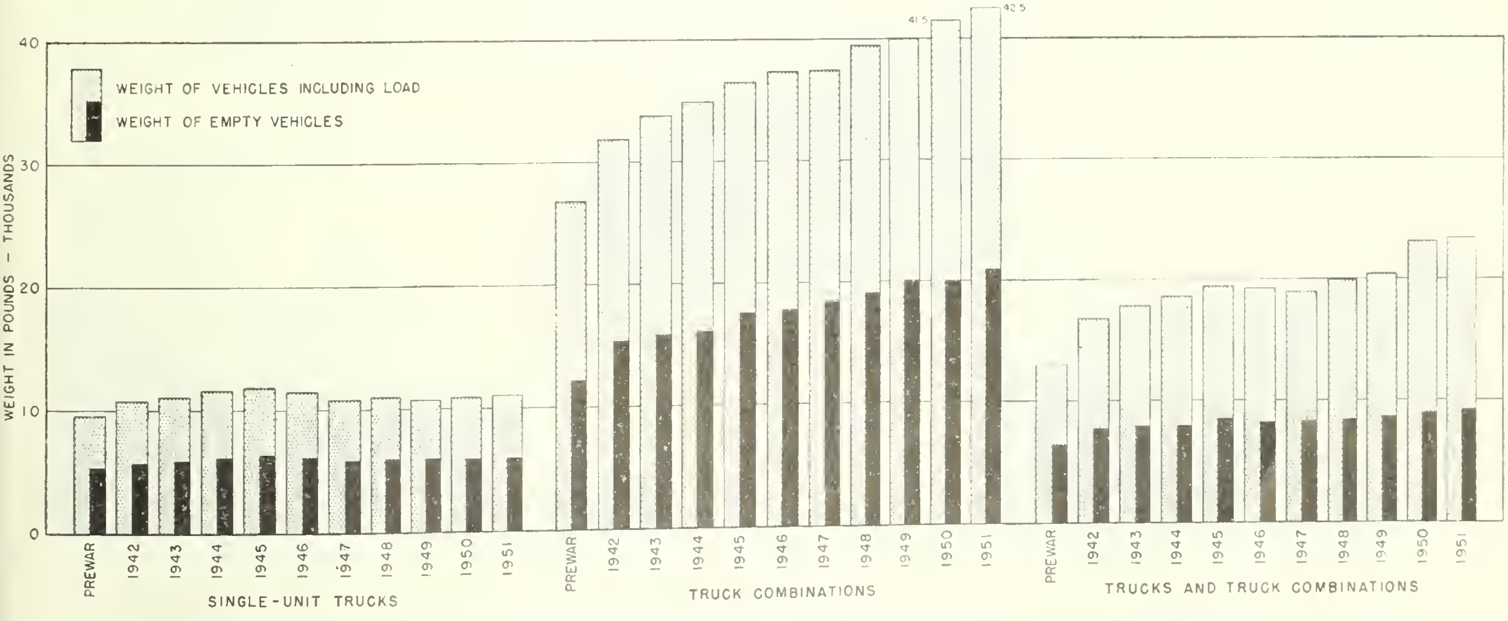

Figure 4.—Average weights of loaded and empty trucks and truck combinations in the summers of 1942-51 and a prewar year.

Table 4.—Average weight (in pounds) of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1951

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Average Weights of Loaded Vehicles													
Single-unit trucks:													
Panel and pick-up	5,072	5,312	4,876	5,081	5,239	5,336	5,159	7,162	5,898	5,154	4,432	4,750	5,498
Other 2-axle, 4-tire	6,522	6,719	6,785	6,717	6,623	6,734	7,233	6,746	6,898	6,877	5,674	5,910	6,522
Other 2-axle, 6-tire	14,776	15,813	13,007	14,411	13,572	14,163	14,755	13,693	14,031	14,378	12,899	13,430	14,069
3-axle	31,746	37,544	27,268	32,397	27,449	26,764	28,378	28,217	27,700	32,186	29,302	29,717	29,924
Average	11,873	13,048	10,796	11,892	10,754	11,089	11,123	9,897	10,691	10,078	10,890	10,598	11,024
Truck combinations:													
Truck-tractor and semitrailer	40,212	42,705	39,128	40,771	39,730	35,052	41,548	39,135	39,399	46,153	50,656	49,445	41,373
Truck and trailer	(1)	(1)	(1)	53,856	66,785	(1)	32,343	59,364	58,778	65,143	57,397	58,601	58,599
Average	40,199	42,810	39,005	40,814	41,315	35,061	41,115	39,912	40,143	48,785	52,297	51,439	42,501
Average, all trucks and combinations	21,014	24,572	22,060	23,062	25,821	18,190	20,469	20,955	22,254	22,151	29,213	26,992	23,376
Average Weights of Empty Vehicles													
Single-unit trucks:													
Panel and pick-up	4,082	4,109	3,822	3,927	4,065	4,120	4,119	5,010	4,425	4,026	3,861	3,959	4,210
Other 2-axle, 4-tire	4,997	4,652	5,309	4,894	4,907	4,629	5,162	5,232	5,032	5,223	4,523	4,770	4,924
Other 2-axle, 6-tire	8,858	8,865	7,359	8,160	8,131	7,926	8,275	8,151	8,120	8,031	8,292	8,153	8,137
3-axle	14,707	17,174	13,705	15,297	12,896	15,169	12,819	18,959	14,257	16,696	16,231	16,356	15,187
Average	6,599	6,738	5,276	5,930	5,990	5,806	6,081	6,063	5,622	5,442	6,080	5,730	5,937
Truck combinations:													
Truck-tractor and semitrailer	21,838	21,033	19,423	20,360	19,719	18,991	21,256	19,343	19,770	24,742	23,509	24,179	20,282
Truck and trailer	(1)	(1)	(1)	19,763	26,442	(1)	18,778	52,760	28,648	30,734	27,668	28,414	28,325
Average	21,777	21,037	19,417	20,357	20,529	18,992	21,114	20,402	20,360	26,033	25,605	25,789	20,911
Average, all trucks and combinations	9,559	10,607	8,069	9,167	10,555	7,933	9,024	8,944	8,910	8,407	10,252	9,280	9,197

[1] Data omitted because of insufficient sample.

creased in New England and Middle Atlantic, decreased in East North Central, East South Central, West North Central, and Mountain regions and remained about the same in other regions.

The increase in travel by all types of passenger vehicles amounted to 11 percent compared to only 2 percent for freight-carrying vehicles. This small increase in truck and truck combination traffic is particularly noteworthy when it is noted that there was an extremely large increase in that traffic in 1950 compared to the previous year. The increase in travel by passenger cars was 7 percent while the increase in travel by all types of freight-carrying vehicles amounted to 18 percent, the increase being 12 percent for single-unit trucks and 33 percent for truck combinations. Thus the 1951 figures show that the extremely rapid rate of increase in truck traffic which occurred from 1949 to 1950 was not continued into 1951. However, such data as are available for 1952 indicate that this leveling off may have been temporary, and that truck traffic may again be increasing faster than passenger car traffic.[5]

The percentage of travel by vehicle types on main rural roads in 1951 is given in table

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[5] Preliminary data from Alabama, Connecticut, Delaware, Maine, Missouri, Nevada, North Carolina, Oregon, and Vermont show a 13 percent increase in travel by trucks and combinations in 1952 compared to 1951.

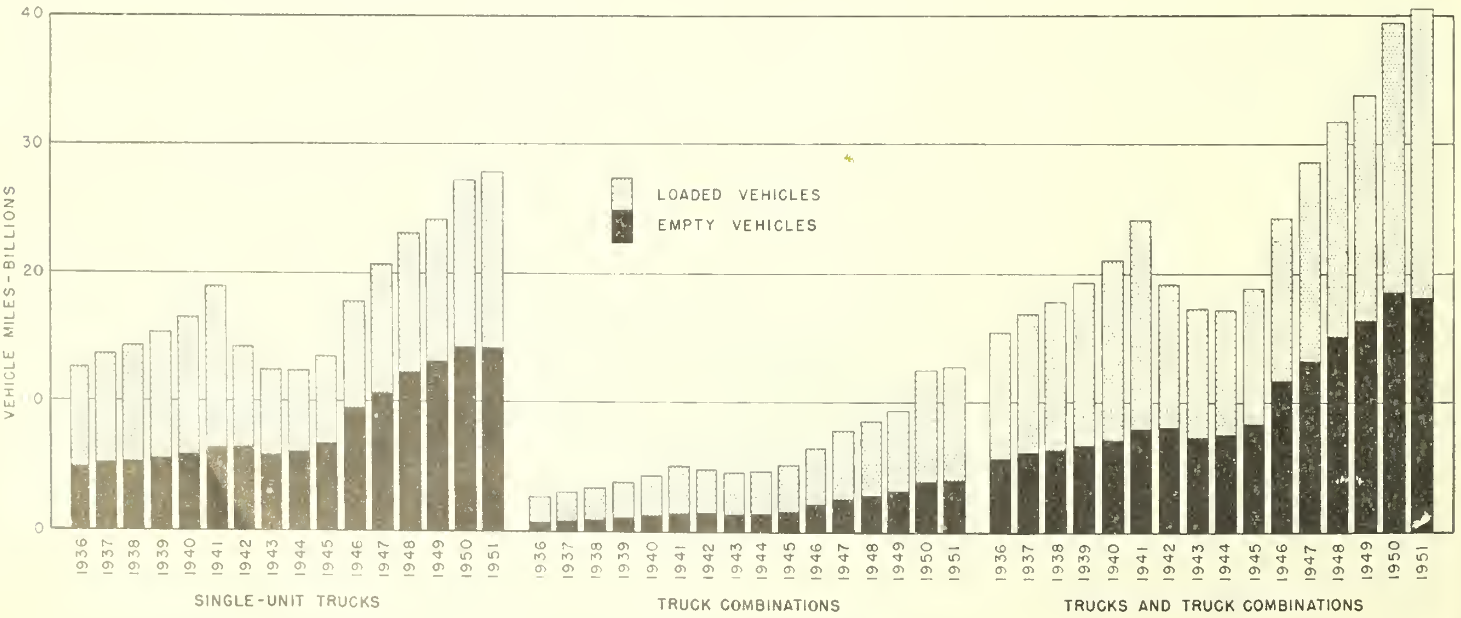
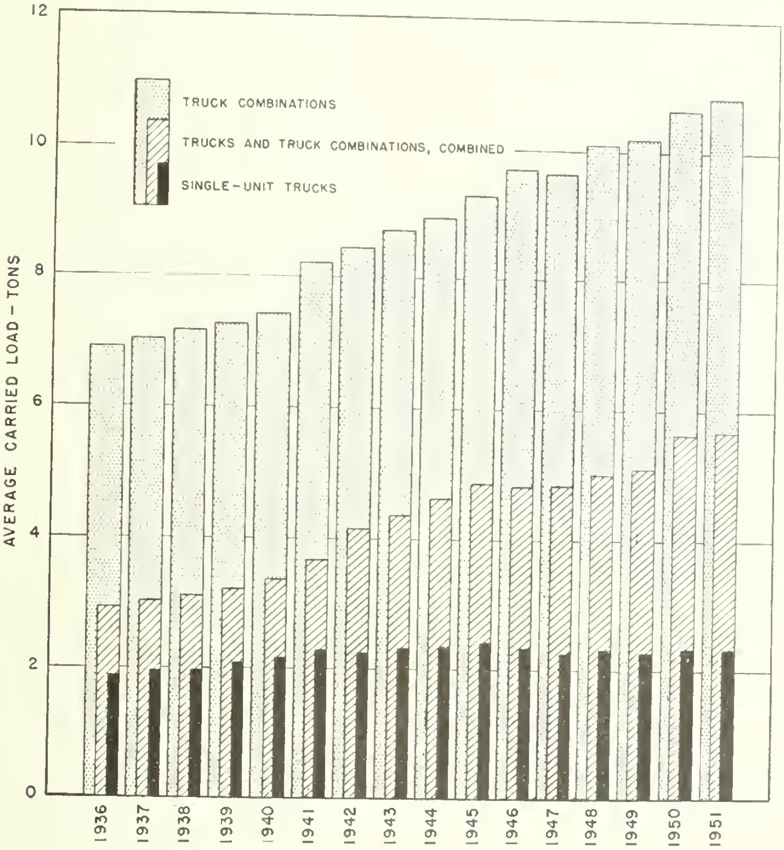


Figure 5.—Travel on main rural roads, 1936–51, by loaded and by empty trucks and truck combinations.

Table 5.—Comparison of estimated vehicle-miles of travel on main rural roads in 1936, 1941, 1946, 1950, and 1951

Year	All vehicles, vehicle-miles	Passenger cars and busses[1]		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*		*Millions*
1936	88,412	82.6	73,005	17.4	15,407	82.1	12,650	17.9	2,757
1941	122,505	80.3	98,320	19.7	24,185	78.8	19,057	21.2	5,128
1941:1936 ratio	*1.39*	*.97*	*1.35*	*1.13*	*1.57*	*.96*	*1.51*	*1.18*	*1.86*
1946	124,149	80.4	99,803	19.6	24,346	73.3	17,838	26.7	6,508
1946:1941 ratio	*1.01*	*1.00*	*1.02*	*.99*	*1.01*	*.93*	*.94*	*1.26*	*1.27*
1946:1936 ratio	*1.40*	*.97*	*1.37*	*1.13*	*1.58*	*.89*	*1.41*	*1.49*	*2.36*
1950	174,349	77.2	134,528	22.8	39,821	68.4	27,256	31.6	12,565
1951	189,651	78.6	149,110	21.4	40,541	68.6	27,810	31.4	12,731
1951:1950 ratio	*1.09*	*1.02*	*1.11*	*.94*	*1.02*	*1.00*	*1.02*	*.99*	*1.01*
1951:1941 ratio	*1.55*	*.98*	*1.52*	*1.09*	*1.68*	*.87*	*1.46*	*1.48*	*2.48*
1951:1936 ratio	*2.15*	*.95*	*2.04*	*1.23*	*2.63*	*.84*	*2.20*	*1.75*	*4.62*

[1] Percentages of total 1951 travel by passenger cars and by busses are reported separately in table 3.



igure 6.—Average load carried by trucks and truck combinations on main rural roads, 1936–51.

3. In this table all single-unit trucks are ivided into classification types based on he axle and tire arrangements, while the ruck combinations are classified according o the total number of axles of the combination. The classification of vehicles into hese types has been used in the last five nnual surveys. It has several advantages ver the original "light, medium, and heavy" rouping, particularly in that it provides ore homogeneous groupings and more posiive identification of the types. It is regrettable that no direct comparison can be made by vehicle types between the old and the new classifications, or between data collected in 1946 and earlier years with such data collected in 1947 and thereafter, but the convenience and advantages of the new system outweigh the disadvantages caused by the change.

The data in table 3 indicate that truck and truck combination travel in 1951 was more than 20 percent of the total travel in all but the New England and Pacific regions. It was between 20 and 25 percent in all remaining regions except the East South Central region where it was over 28 percent.

A comparison with the same table in the 1950 report shows that the proportion of trucks was lower in 1951 than in 1950 in every region except the Pacific region where the proportion of freight-carrying vehicles increased slightly.

Table 3 indicates also that the usage of certain types of freight-carrying vehicles varies in different sections. For instance, the truck and trailer combinations with six or more axles and the truck-tractor and semitrailer with five or more axles are used far more frequently in the Pacific region than in any other area. Truck and trailer combinations are used much less in the East South Central region and in the three eastern regions than in other sections. The percentage of combination-type vehicles, nation-wide, was 6.71 percent, a slight decrease from the 1950 figure of 7.21 percent but exceeding the percentage figures of 5.95 in 1949, 5.84 in 1948, 5.73 in 1947, and 5.26 in 1946.

The average weights of loaded and empty trucks and truck combinations, separately and combined, are shown graphically in figure 4 for each year from 1942 to 1951, inclusive, and for a prewar year, generally 1936 or 1937. The weights of single-unit trucks, both loaded and empty, increased each year from the 1936–37 period through 1945, then decreased slightly and leveled off around 11,000 pounds for loaded vehicles and slightly less than 6,000 pounds for empty vehicles. At the same time weights of truck combinations, both loaded and empty, have increased each year during the

Table 6.—Comparison of estimated percentage of trucks and truck combinations loaded, average carried load, and ton-miles carried on main rural roads in 1936, 1941, 1946, 1950, and 1951

Year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1936	62.8	2.90	28,005	60.7	1.86	14,258	72.2	6.90	13,747
1941	66.7	3.64	58,737	65.4	2.29	28,487	71.6	8.23	30,250
1941:1936 ratio	*1.06*	*1.26*	*2.10*	*1.08*	*1.23*	*2.00*	*.99*	*1.19*	*2.20*
1946	51.7	4.84	60,892	46.4	2.31	19,101	66.2	9.70	41,791
1946:1941 ratio	*.78*	*1.33*	*1.04*	*.71*	*1.01*	*.67*	*.92*	*1.18*	*1.38*
1946:1936 ratio	*.82*	*1.67*	*2.17*	*.76*	*1.24*	*1.34*	*.92*	*1.41*	*3.04*
1950	53.9	5.64	121,091	47.2	2.31	29,645	68.5	10.62	91,446
1951	55.1	5.66	126,402	48.8	2.31	31,396	68.9	10.83	95,006
1951:1950 ratio	*1.02*	*1.00*	*1.04*	*1.03*	*1.00*	*1.06*	*1.01*	*1.02*	*1.04*
1951:1941 ratio	*.83*	*1.55*	*2.15*	*.75*	*1.01*	*1.10*	*.96*	*1.32*	*3.14*
1951:1936 ratio	*.88*	*1.95*	*4.51*	*.80*	*1.24*	*2.20*	*.95*	*1.57*	*6.91*

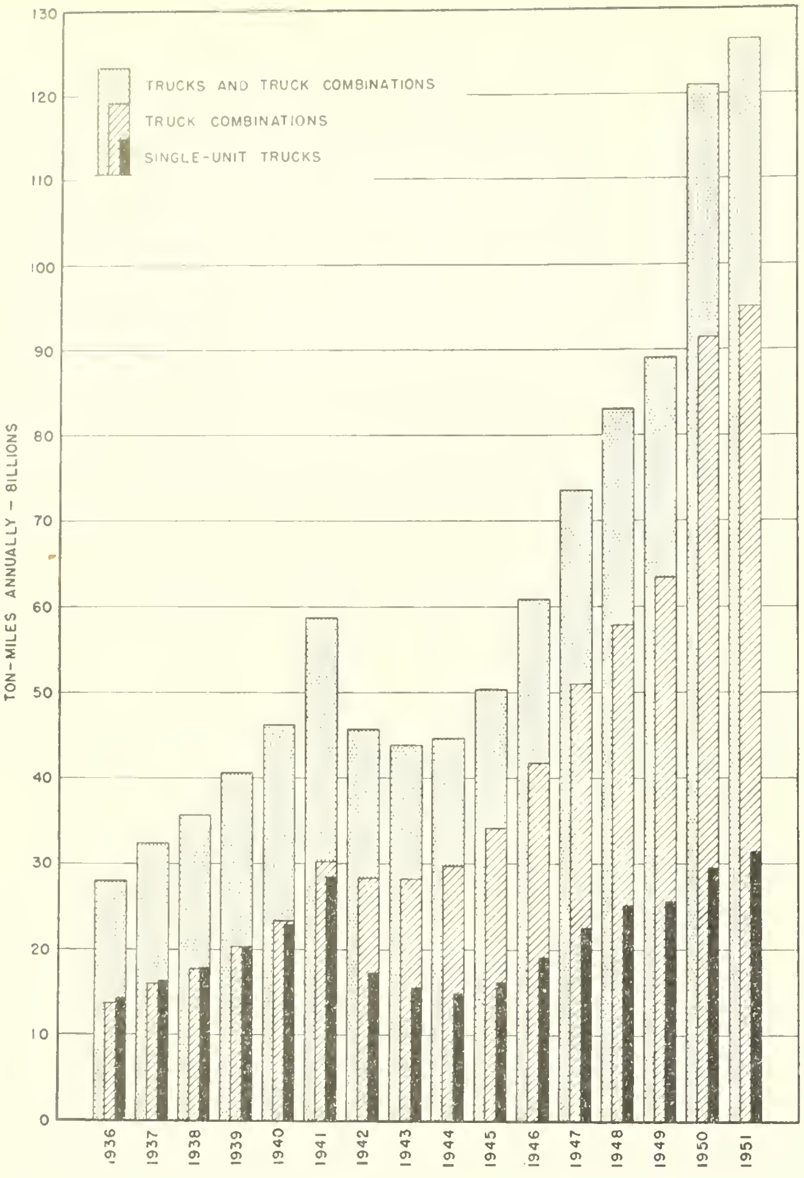


Figure 7.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936-51.

period shown. The increase in average weight of loaded combinations from the 1936–37 period to 1951 was over 58 percent, compared to 12 percent for single-unit trucks.

The increase for all loaded trucks and truck combinations combined was slightly more than 80 percent. It will be noted that the average weight of the loaded single-unit trucks was somewhat less than twice the average weight of the empty vehicles of this type, while the average weight of the loaded combinations was just about twice the average weight of the empty combinations. In the case of the vehicles of both types combined, the loaded vehicles included a higher proportion of combinations than the empty vehicles, since combinations are more often loaded and the average weight of the loaded trucks and combinations was therefore considerably more than twice the average weight of the empty vehicles of both types.

The average weights of the various types of loaded and empty trucks and truck combinations in the summer of 1951 are shown in table 4 for the different regions. This table brings out clearly the important differences that exist in the weight characteristics of the vehicles in the different groups. It will be noted, for example, that for the United States as a whole, the loaded three-axle, single-unit trucks weighed a little more than twice as much as the two-axle, six-tire trucks. The latter, in turn, weighed a little more than twice as much as the two-axle, four-tire trucks. Similar differences existed throughout the various classifications. On the other hand, the regional differences in average weight for each of the vehicle types that are common throughout the country are surprisingly small. The rather low weights of truck and trailer combinations in certain sections of the country, particularly the West North Central region, indicate a predominance of small home-made trailers of low capacity.

Seasonal loadometer data for 1952 and a corresponding period in 1951, received from 9 States as previously referred to, indicate that in 1952 the loaded single-unit trucks were about 4 percent lighter, and the empty trucks about 8 percent lighter, on the average, than in 1951. The loaded tractor-semitrailer combinations were less than 1 percent heavier than in 1951 and the empty combinations about 3 percent heavier. These data indicated no change in the percentage of loaded single-unit trucks, but an appreciable increase in the percentage of truck combinations loaded.

Truck Travel Increases

Figure 5 shows the estimated vehicle-mileage of travel by loaded and empty single-unit trucks and truck combinations, separately and combined, on main rural roads for each year 1936 to 1951, inclusive. This chart demonstrates graphically the steady growth of truck traffic during the prewar years 1936–41, the temporary effect of wartime restrictions in the period 1942–45, and the remarkable increases in truck transportation that have occurred since the end of hostilities in 1945.

Table 5 gives comparisons of the estimated vehicle-mileage of travel by vehicles of different types on all main rural roads

Table 7.—Percentage of vehicle-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried by various types of trucks and truck combinations on main rural roads in 1951 compared to that in corresponding months of 1950

Vehicle type	Percentage of vehicle-miles of travel		Percentage loaded		Average carried load		Percentage of ton-miles carried	
	1951	1950	1951	1950	1951	1950	1951	1950
					Tons	*Tons*		
Single-unit trucks:								
Panel and pick-up	32.76	31.35	39.0	37.4	0.70	0.69	2.86	2.65
Other 2-axle, 4-tire	2.90	2.79	54.6	52.4	.91	.93	.46	.45
Other 2-axle, 6-tire	30.94	32.63	58.0	55.9	3.23	3.20	18.57	19.06
3-axle	2.00	1.68	60.9	58.3	7.53	7.23	2.95	2.32
All single-unit trucks	68.60	68.45	48.8	47.2	2.31	2.31	24.84	24.48
Truck combinations:								
Truck-tractor and semitrailer	29.22	29.43	69.2	68.9	10.48	10.32	67.96	68.87
Truck and trailer	2.18	2.12	65.0	62.3	15.48	15.32	7.20	6.65
All truck combinations	31.40	31.55	68.9	68.5	10.83	10.62	75.16	75.52
All trucks and combinations	100.00	100.00	55.1	53.9	5.66	5.64	100.00	100.00

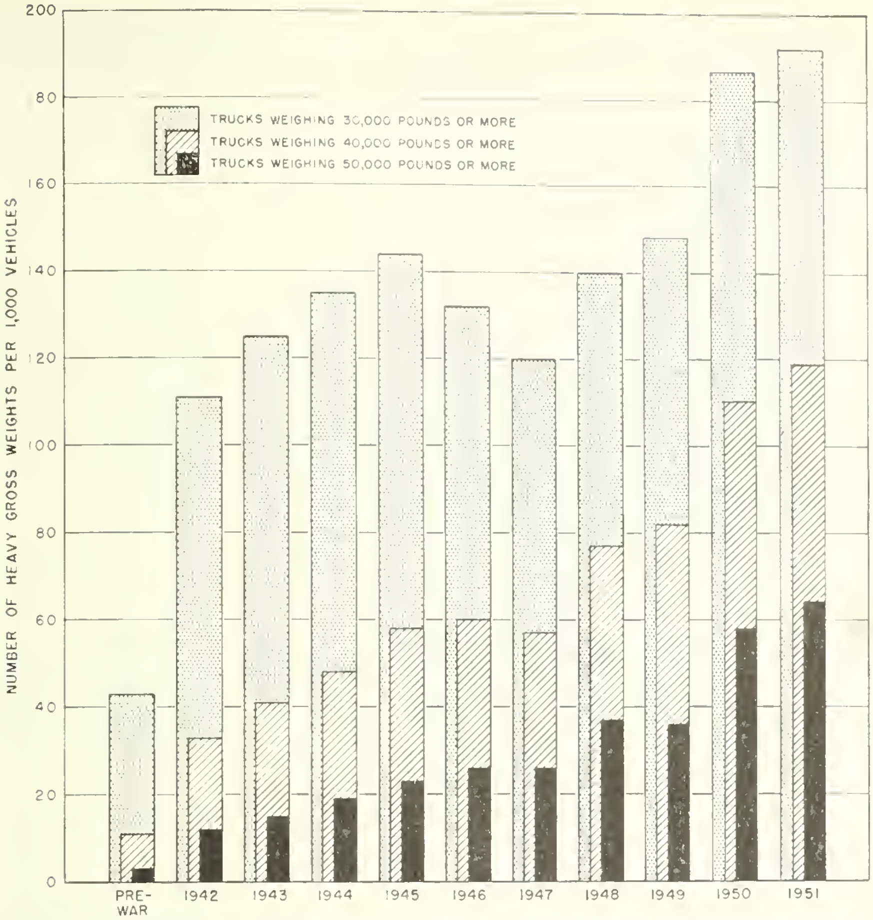


Figure 8.—Number of heavy gross weights per 1,000 trucks and truck combinations (empties included) in the summers of 1942-51 and a prewar year.

in 1936, the earliest year for which comprehensive travel and weight data are available; in 1941, the peak prewar year, 5 years after the beginning of the surveys; in 1946, 10 years after the beginning of the surveys; and in 1950 and 1951 which completes 16 full years of estimates. The ratios of 1951 travel to that of the preceding years indicate that increases for trucks and truck combinations generally were greater than for passenger cars and busses, and that increases for truck combinations were greater than for single-unit trucks. In the 15 years from 1936 to 1951, passenger-car and bus travel combined increased 104 percent, travel by all trucks and combinations increased 163 percent, and travel by truck combinations (considered separately) more than quadrupled, increasing 362 percent.

Volume of Highway Freight

Figure 6 gives a comparison of the average load carried by single-unit trucks and truck combinations, separately and combined, in the 16 years that the planning surveys have been operating. The general trend of load weights was upward throughout the period. The slight decline in the weights of loads carried by single-unit trucks since 1945 has been more than offset by the increased use of combinations and the increased weights of loads carried by vehicles of this type.

Figure 7 shows for each year from 1936 through 1951, the ton-mileage of freight carried by trucks and truck-combinations on main rural roads. The chart demonstrates clearly that truck combinations are transporting each year a larger proportion of the total amount of highway freight. In 1936 the truck combinations hauled slightly less ton-mileage than the single-unit trucks, while in 1950 they hauled more than triple the amount transported by the larger number of lighter vehicles. The rapid rate of annual increase in total freight carried, which took place in 1946 and 1947, was reduced somewhat in 1948 and 1949 to a rate more nearly comparable with that of prewar years. In 1950, however, there was a startling increase in freight ton-mileage somewhat similar to a rise that occurred in 1941. In 1951 the rate of increase was again reduced to one closely comparable with the prewar trend.

In table 6 are shown comparisons of the percentage of vehicles carrying loads, the average carried load, and the ton-mileage carried for single-unit trucks and for truck combinations, separately and combined, in 1951 with corresponding items for other years as in table 5. The trend from 1936 to 1951 of average weight carried, shown graphically in figure 6, and that of the ton-mileage transported during the same period, shown in figure 7, has already been discussed.

The percentage of trucks and truck combinations carrying loads increased notice-

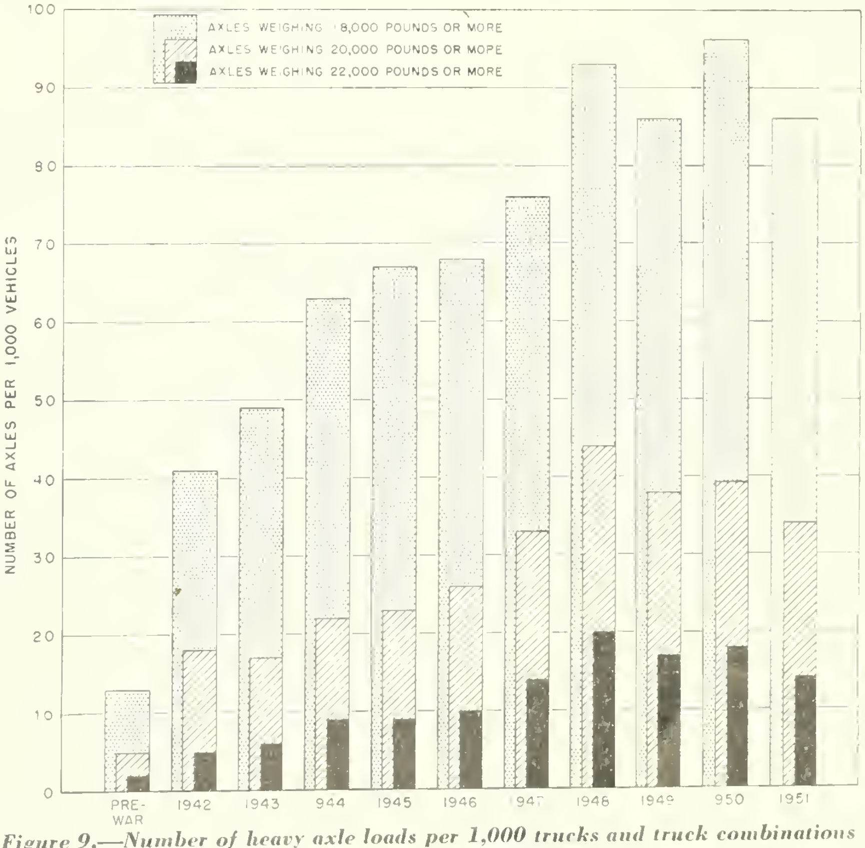


Figure 9.—Number of heavy axle loads per 1,000 trucks and truck combinations (empties included) in the summers of 1942-51 and a prewar year.

Table 8.—Heavy gross weights per 1,000 loaded and empty trucks and truck combinations on main rural roads, summer of 1951

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
	Number per 1,000 Weighing 30,000 Pounds or More												
Single-unit trucks:													
2-axle, 6-tire	15	26	1	13	0	1	(1)	5	1	5	1	3	5
3-axle	295	522	246	374	232	220	325	339	269	356	319	325	323
Average	16	33	8	18	6	4	6	5	5	11	30	22	12
Truck combinations:													
Truck-tractor and semitrailer	541	631	533	577	586	481	600	511	557	645	712	691	580
Truck and trailer	(2)	(2)	(2)	516	679	(2)	317	(2)	506	818	717	735	637
Average	540	631	532	576	594	481	586	493	554	673	713	702	584
Average, all trucks and combinations	153	234	164	191	255	109	158	143	178	164	289	240	192
Comparative average, 1950	137	221	177	189	251	102	142	146	170	160	289	233	187
Comparative average, 1949	117	191	130	153	208	87	139	107	144	118	176	147	148
	Number per 1,000 Weighing 40,000 Pounds or More												
Single-unit trucks:													
2-axle, 6-tire	2	4	0	2	0	(1)	0	0	(1)	0	0	0	1
3-axle	132	292	74	177	46	31	30	89	45	119	52	64	96
Average	5	13	2	7	1	1	1	1	1	3	5	4	3
Truck combinations:													
Truck-tractor and semitrailer	350	425	325	371	345	182	381	310	322	415	560	515	363
Truck and trailer	(2)	(2)	(2)	516	552	(2)	216	(2)	403	551	564	561	496
Average	350	427	325	372	361	183	373	300	326	437	561	526	372
Average, all trucks and combinations	95	152	98	120	154	41	98	85	103	103	216	172	119
Comparative average, 1950	78	135	95	109	140	45	82	79	95	106	214	167	110
Comparative average, 1949	66	120	71	90	105	36	77	54	73	75	121	97	82
	Number per 1,000 Weighing 50,000 Pounds or More												
Single-unit trucks:													
2-axle, 6-tire	0	2	0	1	0	(1)	0	0	(1)	0	0	0	(1)
3-axle	29	49	0	25	9	0	0	3	4	69	0	13	14
Average	1	3	0	1	(1)	(1)	0	(1)	(1)	2	0	1	1
Truck combinations:													
Truck-tractor and semitrailer	117	207	123	159	179	25	205	124	150	296	464	411	187
Truck and trailer	(2)	(2)	(2)	452	510	(2)	208	(2)	374	506	482	486	440
Average	116	210	123	160	204	25	205	120	162	330	469	430	204
Average, all trucks and combinations	31	72	36	50	87	6	54	34	51	78	178	138	64
Comparative average, 1950	24	63	28	41	78	7	44	34	47	76	176	133	58
Comparative average, 1949	15	52	21	33	48	6	32	18	29	51	99	75	36

[1] Less than 5 per 10,000.
[2] Data omitted because of insufficient sample.

ably from 1950 to 1951 in all regions except the New England region where a slight decrease of this factor was found. In the country as a whole, the percentage loaded increased from 53.9 percent in 1950 to 55.1 percent in 1951. Both for single-unit trucks and for truck combinations, the percentage loaded was higher in 1951 than in 1950 or any year since 1945. However, the loaded proportion continued to be considerably less for the single-unit vehicles and slightly less for the truck combinations than in the prewar surveys.

Table 7 gives a detailed comparison of the percentage of vehicle-miles of travel, percentage of vehicles loaded, average carried load, and percentage of total ton-miles of freight carried by the various types of trucks and truck combinations traveling on main roads in 1950 and 1951. Many interesting comparisons can be made from this table showing the relative importance from a freight-carrying standpoint of different portions of the traffic stream. In 1951, for instance, while panel and pick-up trucks traveled almost 33 percent of the vehicle-mileage, they accounted for less than 3 percent of the ton-mileage. The truck-tractor and semitrailers, on the other hand, traveled about 29 percent of the vehicle-mileage but carried almost 68 percent of the ton-mileage.

From the column in table 7, showing percentage loaded by types, it can be observed that the percentage of vehicles carrying loads tends to increase directly as the size of the vehicle type, extending from light panel and pick-up trucks that are loaded 39 percent of the time to the heavy combinations that are loaded about 69 percent of the time.

Gross Weights Increase

Figure 8 shows by years, from the prewar years (generally 1936 or 1937) to 1951, for the United States as a whole, the frequency of gross weights of 30,000 pounds or more, of 40,000 pounds or more, and 50,000 pounds or more. The chart shows strikingly how the frequency of heavy loads has increased year after year reaching amounts in 1951 considerably above any previous level. In this upward climb the 1951 frequencies surpassed even the astonishing maximum levels established for each weight group in the previous year. For instance, the frequency of the loads of 30,000 pounds or more was 3 percent higher than in 1950 and 30 percent higher than in 1949. The loads of 40,000 pounds or more was 8 percent higher than in 1950 and 45 percent higher than in 1949. The increase in loads of 50,000 pounds or more, however, was even more startling, the frequency being 10 percent above the 1950 figure and almost 78 percent above the 1949 figure. The 30,000-pound loads were over 4 times as frequent as in the prewar year; loads of 40,000 pounds or more were 11 times as frequent; while those of 50,000 pounds or more were almost 22 times as frequent as in the 1936–37 period.

The 1951 gross-weight frequency data by vehicle type and region are presented in table 8. No panels, pick-ups, or other two-axle, four-tire, single-unit trucks were found in the survey weighing as much as 30,000 pounds, so there is no entry for these vehicles in the table, though they are included in the total number of vehicles weighed in computing the frequencies for all trucks and combinations. Heavy gross weights are much more frequent in the Pacific region

Table 9.—Heavy axle loads per 1,000 loaded and empty trucks and truck combinations on main rural roads, summer of 1951

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 18,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	45	67	21	43	15	22	20	23	19	31	21	25	28
3-axle	172	332	58	191	10	53	40	64	33	189	29	58	94
Average	29	48	11	27	8	11	11	8	9	16	11	13	15
Truck combinations:													
Truck-tractor and semitrailer	500	523	278	405	180	166	168	197	180	186	106	131	246
Truck and trailer	(1)	(1)	(1)	339	397	(1)	242	(1)	303	191	84	104	189
Average	499	522	278	405	197	166	171	190	187	187	99	124	242
Average, all trucks and combinations	151	207	90	144	88	46	53	60	65	55	44	49	86
Comparative average, 1950	137	208	100	147	98	63	45	67	72	83	69	75	96
Comparative average, 1949	124	195	99	140	89	50	50	51	63	57	37	48	86
NUMBER PER 1,000 WEIGHING 20,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	28	49	7	27	4	6	4	12	6	12	3	6	13
3-axle	104	157	2	81	10	0	0	10	6	123	0	22	37
Average	18	32	3	16	2	3	2	4	3	7	1	4	7
Truck combinations:													
Truck-tractor and semitrailer	321	322	110	221	35	39	43	67	45	56	12	26	99
Truck and trailer	(1)	(1)	(1)	85	26	(1)	13	(1)	19	11	15	14	17
Average	320	320	109	220	34	39	42	64	43	49	13	23	93
Average, all trucks and combinations	97	129	35	79	16	11	12	21	16	17	5	10	34
Comparative average, 1950	82	131	38	80	22	19	12	23	19	35	16	24	39
Comparative average, 1949	73	118	46	78	27	18	12	18	20	26	6	16	38
NUMBER PER 1,000 WEIGHING 22,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	17	31	3	16	2	2	1	4	2	4	1	2	7
3-axle	33	76	0	37	10	0	0	0	4	43	0	8	16
Average	10	19	1	9	1	1	1	1	1	3	(2)	1	4
Truck combinations:													
Truck-tractor and semitrailer	148	182	31	107	10	9	10	10	10	14	5	7	41
Truck and trailer	(1)	(1)	(1)	(1)	11	(1)	0	(1)	7	1	2	2	4
Average	148	180	31	107	10	9	10	9	10	12	4	6	38
Average, all trucks and combinations	46	73	10	39	5	3	3	4	4	5	2	3	14
Comparative average, 1950	39	80	13	42	7	5	3	9	6	16	3	9	18
Comparative average, 1949	33	65	18	39	9	5	3	6	6	11	2	6	17

[1] Data omitted because of insufficient sample.
[2] Less than 5 per 10 000.

than in other parts of the country. In this region 178 of each 1,000 trucks and truck combinations on the main rural highways in 1951, empties included, weighed 50,000 pounds or more and 289 of each 1,000 weighed 30,000 pounds or more. In the East North Central region, 255 of each 1,000 trucks and truck combinations weighed 30,000 pounds or more, almost as many as in the Pacific region, but only 87 of each 1,000 vehicles weighed 50,000 pounds or more, a frequency less than half of that in the Pacific region for this heavy class of vehicle. The lowest frequency of heavy gross loads was found in the East South Central region where only 6 of each 1,000 weighed 50,000 pounds or more, and only 109 of each 1,000 weighed 30,000 pounds or more.

As was pointed out in the discussion of figure 8, the frequencies of heavy gross loads have increased noticeably in the Nation as a whole. This increase is not limited to any certain area but is distributed throughout the entire country. Comparing the frequencies of gross weights in 1951 with those in the previous year, slight decreases were found in the frequencies of gross weights of 30,000 pounds or more in the South Atlantic region and in the West South Central region; in the frequency of gross weights of 40,000 pounds or more in the East South Central and in the Mountain regions; and in the frequency of gross weights of 50,000 pounds or more in the East South Central region. The frequency of gross weights, 50,000 pounds or more, in the West South Central region did not change nor did the frequency of gross weights of 30,000 pounds or more in the Pacific region. The largest increases were found in the New England region where the frequency of loads of 30,000 pounds or more increased from 137 per 1,000 vehicles in 1950 to 153 in 1951. Also in that region, loads of 40,000 pounds or more increased from 78 in 1950 to 95 in 1951, and those of 50,000 pounds or more increased from 24 in 1950 to 31 in 1951 for each 1,000 vehicles.

Seasonal data collected in 1952 and in a corresponding period in 1951 from the 9 States previously listed, indicate that the frequency of the heavier loads is higher in 1952 than in 1951. This increase is particularly noticeable in the frequency of vehicles weighing 50,000 pounds or more which, in the sample received, were 20 percent more frequent in 1952 than in the earlier year. On the other hand, vehicles weighing 40,000 pounds or more were only slightly more frequent in 1952, while the frequency of those weighing 30,000 pounds or more had not changed appreciably.

Frequency of Heavy Axle Loads

Figure 9 shows the frequency of axle loads of 18,000 pounds or more, 20,000 pounds or more, and of 22,000 pounds or more for the prewar years (1936–37) and by years from 1942 to 1951. The frequency of these heavy axle loads increased year by year from the prewar period through 1948. Since 1948, however, the trend apparently has been reversed for, with the exception of 1950, the data seem to indicate a downward trend or at least a leveling off. Such a leveling off in the frequency of the heavier axle loads, though heavy gross loads have increased in the past year, may indicate that more attention is being given to proper load distribution, due perhaps to better enforcement of legal limits.

Table 10.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle, axle-group, or gross-weight legal limits in effect in the States by various percentages (maximum) of overload, summer of 1951

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 6-tire	13	9	6	4	2	(1)
3-axle	72	65	53	15	2	....
Average, single-unit trucks	9	7	5	3	1	(1)
Truck-tractor and semitrailer	114	73	46	16	4	1
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	114	73	46	16	4	1
Average, all trucks and combinations	36	24	16	6	2	(1)
Middle Atlantic:						
2-axle, 6-tire	29	24	19	8	4	1
3-axle	187	152	46	2	....	....
Average, single-unit trucks	22	18	12	4	2	1
Truck-tractor and semitrailer	175	134	89	43	16	2
Truck and trailer	267	69	62	56	....	....
Average, truck combinations	176	133	89	43	16	2
Average, all trucks and combinations	74	57	38	17	7	1
South Atlantic:						
2-axle, 6-tire	9	6	3	1	....	....
3-axle	38	13	6	2	2	....
Average, single-unit trucks	5	3	2	1	(1)	....
Truck-tractor and semitrailer	101	71	50	18	6	(1)
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	101	71	50	18	6	(1)
Average, all trucks and combinations	34	23	16	6	2	(1)
East North Central:						
2-axle, 6-tire	8	3	2	1	....	....
3-axle	34	19	9	....	....	....
Average, single-unit trucks	5	2	1	1	....	....
Truck-tractor and semitrailer	125	65	25	5	1	1
Truck and trailer	221	123	38	8	(1)	(1)
Average, truck combinations	132	69	26	5	1	1
Average, all trucks and combinations	59	30	12	3	(1)	(1)
East South Central:						
2-axle, 6-tire	15	10	7	3	1	(1)
3-axle	56	37	18	7	3	1
Average, single-unit trucks	8	5	4	2	1	(1)
Truck-tractor and semitrailer	129	79	42	15	4	(1)
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	129	79	42	15	4	(1)
Average, all trucks and combinations	35	21	12	5	2	(1)
West North Central:						
2-axle, 6-tire	11	6	4	1	(1)	....
3-axle	63	21	16	5	....	....
Average, single-unit trucks	7	3	2	1	(1)	....
Truck-tractor and semitrailer	201	118	67	23	11	3
Truck and trailer	79	56	19	....	....	....
Average, truck combinations	195	115	65	22	10	3
Average, all trucks and combinations	56	32	19	7	3	1
West South Central:						
2-axle, 6-tire	24	19	14	8	5	3
3-axle	63	33	17	8	....	....
Average, single-unit trucks	9	7	5	3	2	1
Truck-tractor and semitrailer	176	121	78	32	13	2
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	170	117	75	31	13	2
Average, all trucks and combinations	54	38	25	11	5	1
Mountain:						
2-axle, 6-tire	24	15	9	4	2	(1)
3-axle	136	104	80	63	38	12
Average, single-unit trucks	12	8	5	3	2	(1)
Truck-tractor and semitrailer	168	116	74	27	9	2
Truck and trailer	254	130	74	11	2	....
Average, truck combinations	182	118	74	24	8	2
Average, all trucks and combinations	51	33	21	8	3	(1)
Pacific:						
2-axle, 6-tire	6	3	2	1	1	....
3-axle	37	18	2	....	....	....
Average, single-unit trucks	6	3	1	(1)	(1)	....
Truck-tractor and semitrailer	65	33	13	5	(1)	....
Truck and trailer	132	56	26	7	1	....
Average, truck combinations	84	40	17	6	(1)	....
Average, all trucks and combinations	36	17	7	2	(1)	....
United States average:						
2-axle, 6-tire	15	10	7	3	2	(1)
3-axle	71	46	19	6	3	1
Average, single-unit trucks	9	6	4	2	1	(1)
Truck-tractor and semitrailer	139	89	52	20	7	1
Truck and trailer	157	78	32	7	1	(1)
Average, truck combinations	140	88	51	19	7	1
Average, all trucks and combinations	50	32	19	7	3	(1)
Comparative average, 1950	67	44	27	11	5	1
Comparative average, 1949	51	35	23	10	4	1

[1] Less than 5 per 10,000

Table 11.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the permissible load limits recommended by the A.A.S.H.O. by various percentages (maximum) of overload in the summer of 1951

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 6-tire	43	35	29	18	10	4
3-axle	89	76	58	33	17	10
Average, single-unit trucks	25	21	17	10	6	2
Truck-tractor and semitrailer	269	232	194	112	48	9
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	268	231	194	112	48	9
Average, all trucks and combinations	88	76	63	37	17	4
Middle Atlantic:						
2-axle, 6-tire	55	52	47	31	19	7
3-axle	243	201	141	38	13	....
Average, single-unit trucks	38	35	30	18	11	4
Truck-tractor and semitrailer	302	255	203	125	71	16
Truck and trailer	342	191	130	7	....	....
Average, truck combinations	302	255	202	124	70	16
Average, all trucks and combinations	127	109	88	54	31	8
South Atlantic:						
2-axle, 6-tire	16	13	8	2	(1)	....
3-axle	83	54	30	4	2	....
Average, single-unit trucks	10	7	4	1	(1)	....
Truck-tractor and semitrailer	180	133	94	37	10	1
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	180	133	94	37	10	1
Average, all trucks and combinations	61	44	31	12	3	(1)
East North Central:						
2-axle, 6-tire	10	5	3	2	....	....
3-axle	19	6	....	....	....	....
Average, single-unit trucks	6	3	2	1	....	....
Truck-tractor and semitrailer	211	136	72	15	5	1
Truck and trailer	406	374	308	171	33	4
Average, truck combinations	226	154	90	27	7	1
Average, all trucks and combinations	99	67	39	12	3	(1)
East South Central:						
2-axle, 6-tire	15	10	7	3	1	(1)
3-axle	44	30	12	....	....	....
Average, single-unit trucks	8	5	4	1	(1)	(1)
Truck-tractor and semitrailer	113	70	34	12	3	(1)
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	113	70	34	12	3	(1)
Average, all trucks and combinations	31	19	11	3	1	(1)
West North Central:						
2-axle, 6-tire	11	6	4	1	(1)	....
3-axle	38	8	5	3	....	....
Average, single-unit trucks	6	3	2	1	(1)	....
Truck-tractor and semitrailer	173	106	63	18	8	2
Truck and trailer	58	56	19	....	....	....
Average, truck combinations	167	103	61	17	8	2
Average, all trucks and combinations	48	29	17	5	2	1
West South Central:						
2-axle, 6-tire	24	19	15	8	5	3
3-axle	72	33	25	17	8	....
Average, single-unit trucks	9	7	5	3	2	1
Truck-tractor and semitrailer	165	113	68	27	11	1
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	159	109	66	26	11	1
Average, all trucks and combinations	51	36	22	9	5	1
Mountain:						
2-axle, 6-tire	24	15	9	4	2	(1)
3-axle	137	109	74	56	38	12
Average, single-unit trucks	12	8	5	3	2	(1)
Truck-tractor and semitrailer	199	149	103	46	13	2
Truck and trailer	314	211	123	29	2	....
Average, truck combinations	218	159	106	43	11	2
Average, all trucks and combinations	59	43	28	12	4	(1)
Pacific:						
2-axle, 6-tire	6	3	2	1	1	....
3-axle	41	18	2	....	....	....
Average, single-unit trucks	6	3	1	(1)	(1)	....
Truck-tractor and semitrailer	154	112	71	18	5	(1)
Truck and trailer	278	173	85	10	3	1
Average truck combinations	189	129	75	16	4	(1)
Average, all trucks and combinations	75	51	29	6	2	(1)
United States average:						
2-axle, 6-tire	21	17	13	7	4	1
3-axle	83	57	35	12	5	1
Average, single-unit trucks	12	9	7	4	2	(1)
Truck-tractor and semitrailer	198	142	94	40	17	3
Truck and trailer	282	211	139	54	10	2
Average, truck combinations	204	147	97	41	17	3
Average, all trucks and combinations	72	52	35	16	7	1
Comparative average, 1950	91	68	46	21	10	3
Comparative average, 1949	68	53	38	19	10	2

[1] Less than 5 per 10,000.

Table 9 gives data concerning the number of heavy axle loads per 1,000 loaded and empty trucks and truck combinations of various types on the main rural roads by regions in 1951. Since no panel or pick-up trucks were found with axles weighing 18,000 pounds or more there is no entry for these in the table though they are included in figuring the frequencies for all trucks and truck combinations.

Though the greatest frequency of heavy gross weights was in the Pacific region, as shown in table 8, the lowest frequency of heavy axle loads was in that region. Frequencies almost as low were found in the West North Central and the East South Central regions. In the Pacific region, only 2 axles of 22,000 pounds or more were found in 1951 for each 1,000 vehicles while in each of the two Central regions mentioned, only 3 such axles were found for each 1,000 vehicles weighed. By far the greatest frequency of heavy axle loads was in the Middle Atlantic region and the next

Table 12.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle-group loads recommended by the A.A.S.H.O. by various percentages of overload in the summer of 1951

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than— 5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 6-tire	(1)	......	......	......	......	......
3-axle	75	60	49	27	15	......
Average, single-unit trucks	2	2	1	1	(1)	......
Truck-tractor and semitrailer	64	40	26	7	2	1
Truck and trailer	......	......	......	......	......	......
Average, truck combinations	64	40	26	7	2	1
Average, all trucks and combinations	18	12	8	3	1	(1)
Middle Atlantic:						
2-axle, 6-tire	4	3	3	1	1	......
3-axle	232	186	141	35	13	......
Average, single-unit trucks	11	8	7	2	1	......
Truck-tractor and semitrailer	134	105	72	29	19	4
Truck and trailer	260	62	62	7	......	......
Average, truck combinations	135	105	72	29	19	4
Average, all trucks and combinations	53	41	29	11	7	1
South Atlantic:						
2-axle, 6-tire	(1)	(1)	(1)	......	......	......
3-axle	67	40	25	1	......	......
Average, single-unit trucks	2	1	1	(1)	......	......
Truck-tractor and semitrailer	68	48	34	16	4	(1)
Truck and trailer	......	......	......	......	......	......
Average, truck combinations	68	48	34	16	4	(1)
Average, all trucks and combinations	22	15	11	5	1	(1)
East North Central:						
2-axle, 6-tire	......	......	......	......	......	......
3-axle	9	3	......	......	......	......
Average, single-unit trucks	(1)	(1)	......	......	......	......
Truck-tractor and semitrailer	137	84	45	7	2	(1)
Truck and trailer	384	359	294	164	33	4
Average, truck combinations	156	105	64	19	4	(1)
Average, all trucks and combinations	66	45	27	8	2	(1)
East South Central:						
2-axle, 6-tire	(1)	(1)	(1)	(1)	(1)	......
3-axle	10	10	6	......	......	......
Average, single-unit trucks	(1)	(1)	(1)	(1)	(1)	......
Truck-tractor and semitrailer	11	6	2	1	(1)	......
Truck and trailer	......	......	......	......	......	......
Average, truck combinations	11	6	2	1	(1)	......
Average, all trucks and combinations	2	1	(1)	(1)	(1)	......
West North Central:						
2-axle, 6-tire	......	......	......	......	......	......
3-axle	27	5	5	3	......	......
Average, single-unit trucks	1	(1)	(1)	(1)	......	......
Truck-tractor and semitrailer	116	74	43	12	4	1
Truck and trailer	58	19	12	......	......	......
Average, truck combinations	113	71	41	11	4	1
Average, all trucks and combinations	30	19	11	3	1	(1)
West South Central:						
2-axle, 6-tire	(1)	......	......	......	......	......
3-axle	46	25	25	17	8	8
Average, single-unit trucks	(1)	(1)	(1)	(1)	(1)	(1)
Truck-tractor and semitrailer	84	56	29	9	5	1
Truck and trailer	......	......	......	......	......	......
Average, truck combinations	81	54	28	9	5	1
Average, all trucks and combinations	23	15	8	3	1	(1)
Mountain:						
2-axle, 6-tire	......	......	......	......	......	......
3-axle	130	103	74	53	25	......
Average, single-unit trucks	3	3	2	1	1	......
Truck-tractor and semitrailer	151	117	87	39	11	1
Truck and trailer	254	176	114	25	2	......
Average, truck combinations	168	127	91	37	10	1
Average, all trucks and combinations	41	32	23	9	3	(1)
Pacific:						
2-axle, 6-tire	......	......	......	......	......	......
3-axle	35	9	2	......	......	......
Average, single-unit trucks	3	1	(1)	......	......	......
Truck-tractor and semitrailer	142	107	69	17	5	(1)
Truck and trailer	253	158	74	7	3	1
Average, truck combinations	174	122	70	14	4	(1)
Average, all trucks and combinations	68	47	27	5	2	(1)
United States average:						
2-axle, 6-tire	1	(1)	(1)	(1)	(1)	......
3-axle	71	47	33	11	4	(1)
Average, single-unit trucks	3	1	1	(1)	(1)	(1)
Truck-tractor and semitrailer	107	73	45	14	6	1
Truck and trailer	256	191	127	50	10	2
Average, truck combinations	117	81	51	16	6	1
Average, all trucks and combinations	39	26	17	5	2	(1)
Comparative average, 1950	44	33	22	8	3	1
Comparative average, 1949	28	21	14	7	3	1

[1] Less than 5 per 10.000.

greatest in New England. In these two regions the relatively high frequency is attributable mainly to the large number of two-axle truck-tractors pulling one-axle or two-axle semitrailers. The relative infrequency of heavy axles in the Pacific region, in the presence of a large proportion of heavy gross loads, indicates a better distribution of the loads over a larger number of axles.

Although the frequency of heavy gross loads has increased somewhat in all regions, as stated in connection with the discussion of table 8, the trend in frequency of heavy axle loads followed a different pattern. This is demonstrated by comparing the frequency of heavy axle loads in 1951 with those of 1950, as shown for each region in table 9, and noting that the frequency of heavy axle loads decreased in every region except the New England and West North Central regions, whereas table 8 shows that the frequency of gross loads increased somewhat in most regions.

Seasonal information collected in 1952 and in a corresponding period in 1951, reported by the 9 States previously mentioned, indicates that the frequency of the heavier axle loads may generally be higher in 1952 than in the previous year. However, the data so far available are not sufficient to conclude that the leveling off in the trend of heavy axle load frequencies noted during the preceding few years is temporary only.

Loads Above Legal Limits

Table 10 shows the number of trucks and truck combinations of each type, per 1,000 such vehicles counted, empties included, that exceeded the legal axle, axle-group, or gross-weight limits in effect in the individual States in the summer of 1951, and the number per 1,000 that exceeded these limits by various percentages. Comparative figures are given at the bottom of the table for the Nation as a whole, for 1949 and 1950. It shows that, on the whole, there was much better compliance with legal limitations in 1951 than in 1950 and somewhat better than in 1949. Only three-quarters as many vehicles were overloaded in 1951 as in the previous year, and only two-thirds as many exceeded the State weight limits by 20 percent or more.

Loads in excess of State law were, in 1951, most frequent in the Middle Atlantic States, although even in this region, compared to that of 1950, a slight reduction in the `frequency of overloaded vehicles was found. Slight increases in the frequency of overloaded vehicles were found in the New England and in the West North Central regions, but substantial decreases were recorded in all other areas. In the East South Central region, where in 1950 extraordinary conditions in one State caused the highest regional frequency (115 per 1,000) of overloading to be recorded, the 1951 data indicated a frequency of only 35 overloaded vehicles per 1,000 counted. This 1951 frequency in the East South Central region was next to the lowest figure recorded in that year, it being bettered only by the South Atlantic region with a frequency of only 34 excessively loaded vehicles for each 1,000. After the Middle Atlantic region where, of all loaded and empty trucks and truck combinations weighed in 1951, 74 exceeded one or more of the State weight limits, the East North Central region had the second highest rate of overloads (59) and in descending order of rates of violation were the West North Central (56), the West South Central (54), the Mountain (51), the New England (36), the Pacific (36), the East South Central (35), and the South Atlantic (34).

No panel or pick-up truck or other 2-axle, 4-tired truck was weighed that exceeded any of the State weight regulations and these classifications are omitted from tables 10-12 although the number of such vehicles counted is included in the calculations.

Recommended Weight Limits

Uniform regulations concerning maximum allowable gross weights, axle weights, and axle-group weights have been adopted as a policy by the American Association of State Highway Officials and recommended to the State governments for adoption.[6] This policy recommends that no axle shall carry a load in excess of 18,000 pounds and no group of axles shall carry a load in excess of amounts specified in a table of permissible weights based on the distance between the extremes of any group of axles.

The frequencies of axle loads of 18,000 pounds or more, 20,000 pounds or more, and 22,000 pounds or more have already been discussed in connection with table 9 and will not be discussed further here.

As might be expected, many vehicles were so loaded that they exceeded more than one recommended weight limit, and some vehicles had more than one axle loaded in excess of the recommended limit. Counting each vehicle only once, regardless of the number of ways in which it exceeded any of the A.A.S.H.O. recommended limits, table 11 was prepared to show the number of vehicles per 1,000 of each type, both loaded and empty, that exceeded the limits by various percentages. Those vehicles which exceeded more than one provision of the recommended restrictions were tabulated only in the column showing the highest percentage excess of any item.

In the various regions of the United States, the number of vehicles out of every 1,000 that exceeded the recommendations in 1951 was lower in each region except in the West North Central where the frequency of all excessive loads remained the same as in 1950. As might be expected, due to the high frequency of excessively heavy axles in the Middle Atlantic region, as indicated in table 9, that region led all others in the number of vehicles out of every 1,000 that exceeded any of the A.A.S.H.O. recommendations (127 for each 1,000 vehicles counted), while in descending order were the East North Central (99), New England (88), Pacific (75), South Atlantic (61), Mountain (59), West South Central (51), West North Central (48), and East South Central (31).

In the United States as a whole, 72 vehicles out of every 1,000 were overloaded to some degree according to the A.A.S.H.O. standards and 16 out of every 1,000 exceeded some one of the recommended provisions by more than 20 percent. The frequency of vehicles exceeding the recommendations by any amount in 1951 was 21 percent less than in 1950. The frequency exceeding the recommendations by more than 20 percent in 1951 was almost 24 percent less than in the previous year.

Table 12 shows the number of vehicles of various types, per 1,000 vehicles with an axle-group load in excess of the limits recommended by the A.A.S.H.O. and in excess of the limits by various percentages. For the United States as a whole, the frequency of axle-group loads in 1951 was lower than in 1950 though somewhat higher than in 1949. When it is considered that the average weight of all trucks and combinations in 1951 exceeded the average weight of these vehicles in 1950 and that, at the same time, the frequency of axle-group loads in excess of the A.A.S.H.O. recommendations decreased materially, it appears that some effort is being made to reduce load concentrations. The 1951 frequency of excessive axle-group loads in the United States as a whole decreased from the frequencies of 1950 in all regions except in the South Atlantic and the West North Central regions. As in the previous year, the highest frequency of excessive axle-group loads was found in the Pacific region (68 per 1,000 vehicles), while the regions in descending order of the number of vehicles with excessive axle-group loads were East North Central (66), Middle Atlantic (53), Mountain (41), West North Central (30), West South Central (23), South Atlantic (22), New England (18), and East South Central (2).

It will be noted that a higher proportion of the vehicles have excessive axle-group loads in the Pacific region than elsewhere, whereas table 9 shows that this region has the lowest frequency of heavy axle loads. This is because of the widespread use of multiple-axle vehicles in California and neighboring States.

In the United States as a whole, the number of vehicles with excessive axle-group loads per 1,000 counted in 1951 was about 11 percent less than the frequency found in the previous year, and the frequency of those exceeding the recommended amounts by 20 percent or more was about one-third less.

State Limits Higher

In considering the data concerning the frequencies of vehicles exceeding the State legal limits and the A.A.S.H.O. recommendations, especially the frequencies in the Middle Atlantic and New England regions, the fact should be recognized that higher limits generally are permitted under State laws in these areas than are recommended by the Association. Axles exceeding the recommended limits by as much as 25 percent may be within the legal limits of certain States, particularly in these two regions. Some States have no axle-group limits and one State has no prescribed axle-load limit in their motor-vehicle restrictions, a fact that further complicates direct comparison of excess weights based on law and those based on the recommendations. Comparison of the frequency data for New England and the Middle Atlantic regions given in table 11 with those in table 10 shows that only about one-third to one-half of the vehicles exceeding one or more of the Association recommendations actually exceeded a State legal limit. Due to more stringent weight laws in the East South Central, the West North Central, and the West South Central regions, the frequency of vehicles exceeding the State weight limits in the States of these regions is slightly greater than the frequency of those exceeding the Association recommendations. For the United States as a whole, over two-thirds of the vehicles exceeding one or more of the Association recommendations also exceeded a State legal limit.

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[6] *Policy concerning maximum dimensions, weights, and speeds of motor vehicles to be operated over the highways of the United States*, adopted April 1, 1946, by the American Association of State Highway Officials; published by the Association in 1946.

Road Test One—MD

A Motion Picture

Road Test One—MD, a motion picture produced by the Bureau of Public Roads, is now available for lending to interested organizations. The 16-millimeter color and sound film, in three reels, has a running time of about an hour and a quarter. It depicts the operation of and explains the conclusions drawn from the test conducted in 1950 at La Plata, Md., under the direction of the Highway Research Board on behalf of 11 Eastern States and with the cooperation of the Bureau of Public Roads. The final report of the study was recently published by the Board.

In the test, a 1.1-mile section of typical concrete pavement, which had withstood 10 years of weathering and moderate traffic with very slight distress, was subjected to 6 months of continuous controlled truck traffic. On each of four test lanes, pairs of trucks were operated—single-axle trucks with 18,000- and 22,400-pound rear-axle loads, and tandem-axle trucks with 32,000 and 44,800 pounds on the tandem rear axles. Pumping, cracking, deflections and strains, and other evidences of distress and failure were carefully recorded and analyzed in conjunction with data on soils, rainfall, etc.

Part I of the motion picture illustrates the nature and scope of the field operations. Evidence of the effect of the various loadings is shown in a chronological series of scenes focused on typical slabs.

Part II, by means of working models and animated charts, illustrates the data collected and the conclusions derived therefrom. The causes and effects of the phenomena observed are clearly and simply explained in layman's language.

The nature and distribution of the types of soils underlying the pavement, and their relation to pavement failure, are portrayed. The effects of typical rainfalls in the gradual increase of pumping and cracking are demonstrated, followed by a comparison of cracking under the various axle loadings on the uniform basis of lineal feet of cracking per slab on a single type (the predominant A-6) of soil.

A model pavement, supported first by granular soil and then by fine-grained soil, shows how pumping occurs and what are its effects. The model slab, undermined by pumping, is actually broken by a miniature loaded truck. The nature and causes of stresses are demonstrated, and comparisons are made of the magnitudes and locations of stresses caused by single-axle and tandem-axle trucks, on both fully supported and pumped-out subgrades. The cause of longitudinal cracking is demonstrated.

The film ends with a summary of the conclusions drawn from the test, and brief comment on their significance.

Road Test One—MD may be borrowed by any responsible organization, without charge except for the nominal shipping costs, by writing to the Visual Education Section, Bureau of Public Roads, Washington 25, D. C. It is anticipated that there will be considerable demand for this film, and the number of available prints will be limited. In requesting loan of the picture, several alternate dates should be proposed. Loans can be made only for short periods of time.

New Publications

The *Annual Report of the Bureau of Public Roads* for the fiscal year ended June 30, 1952, is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 25 cents a copy.

This 81-page publication discusses the progress that was made during the fiscal year in Federal-aid highway planning, programing, and construction, as well as other functions of the Bureau relating to highways.

Accomplishments of the Bureau of Public Roads during the past fiscal year may be summarized in three main categories: the Federal-aid highway construction program, assistance to foreign countries in highway matters, and the research program in fields relating to highway improvement.

During the year the construction program was carried forward with the $500 million Federal-aid authorization together with State and local government matching funds. Work completed involved the improvement of 5,628 miles of principal intercity routes which carry the predominant portion of all rural traffic, the elimination of traffic-congested arteries in many of our populous cities by constructing 772 miles of modern high-speed expressways and other high-type facilities, and the improvement of 11,109 miles of the more important secondary roads serving the rural population. In addition to the regular Federal-aid program, emergency flood relief projects and improvements in National parks and forests totaled 601 miles. The total length of construction projects completed during the fiscal year ending June 30, 1952, at a cost of $846 million was 18,110 miles.

A second phase of the Bureau's activity provided for aid to foreign countries which involved supervising the equipping, training, and organizing of highway departments, and assisting in the planning and restoring of highway systems. Such assistance was made available to Turkey, Ethiopia, Liberia, and the Philippines. During the year, over 220 engineers from more than 54 countries came to the Bureau for assistance in studying American highway practice.

A third phase of the work of the Bureau related to research in the fields of highway finance and administration, highway transport, hydraulics, and physical studies of the properties of highway materials and soils, and the design of pavements and structures.

REPRINTS

Also available from the Superintendent of Documents are reprints of the articles *Highway Transportation Economics* by Richard M. Zettel, appearing in the August 1952 issue of PUBLIC ROADS, vol. 27, No. 3 (10 cents), and *Recent Trends in Highway Bond Financing* by Messrs. Duzan, McCallum, and Todd, appearing in the October 1952 issue, vol. 27, No. 4 (20 cents).

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Work of the Public Roads Administration:
1941, 15 cents. 1946, 20 cents. 1948, 20 cents.
1942, 10 cents. 1947, 20 cents. 1949, 25 cents.

Public Roads Administration Annual Reports: 1943; 1944; 1945. *(Free from Bureau of Public Roads)*

Annual Reports of the Bureau of Public Roads:
1950, 25 cents. 1951, 35 cents. 1952, 25 cents.

HOUSE DOCUMENT NO. 462

Part 1.—Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2.—Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3.—Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4.—Official Inspection of Vehicles. 10 cents.

Part 5.—Case Histories of Fatal Highway Accidents. 10 cents.

Part 6.—The Accident-Prone Driver. 10 cents.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act. 10 cents.

Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act. 15 cents. *(revised 1952)*

Act III.—Uniform Motor-Vehicle Civil Liability Act. 10 cents.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act. 15 cents. *(revised 1952)*

Act V.—Uniform Act Regulating Traffic on Highways. 20 cents.

Model Traffic Ordinance. 15 cents. *(revised 1952)*

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.

United States System of Numbered Highways together with the Federal-Aid Highway System (also shows in color National forests, parks, and other reservations). 5 by 7 feet (in 2 sheets), scale 1 inch equals 37 miles. $1.25.

United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

MISCELLANEOUS PUBLICATIONS

Bibliography of Highway Planning Reports. 30 cents.

Construction of Private Driveways (No. 272MP). 10 cents.

Economic and Statistical Analysis of Highway Construction Expenditures. 15 cents.

Electrical Equipment on Movable Bridges (No. 265T). 40 cents.

Factual Discussion of Motortruck Operation, Regulation, and Taxation. 30 cents.

Federal Legislation and Regulations Relating to Highway Construction. 40 cents.

Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.

Guides to Traffic Safety. 10 cents.

Highway Accidents. 10 cents.

Highway Bond Calculations. 10 cents.

Highway Bridge Location. (No. 1486D). 15 cents.

Highway Capacity Manual. 65 cents.

Highway Needs of the National Defense (House Document No. 249). 50 cents.

Highway Practice in the United States of America. 75 cents.

Highway Statistics (annual):
1945, 35 cents. 1947, 45 cents. 1949, 55 cents.
1946, 50 cents. 1948, 65 cents. 1950, 60 cents.

Highway Statistics, Summary to 1945. 40 cents.

Highways in the United States (*nontechnical*). 15 cents.

Highways of History. 25 cents.

Identification of Rock Types. 10 cents.

Interregional Highways (House Document No. 379). 75 cents.

Legal Aspects of Controlling Highway Access. 15 cents.

Local Rural Road Problem. 20 cents.

Manual on Uniform Traffic Control Devices for Streets and Highways. 75 cents.

Mathematical Theory of Vibration in Suspension Bridges. $1.25.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft. $1.75.

Public Control of Highway Access and Roadside Development. 35 cents.

Public Land Acquisition for Highway Purposes. 10 cents.

Roadside Improvement (No. 191MP). 10 cents.

Selected Bibliography on Highway Finance. 55 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks (FP–41). $1.50.

Taxation of Motor Vehicles in 1932. 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces. 10 cents.

Transition Curves for Highways. $1.25.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States.

Bibliography on Highway Lighting.

Bibliography on Highway Safety.

Bibliography on Land Acquisition for Public Roads.

Bibliography on Roadside Control.

Express Highways in the United States: a Bibliography.

Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.

Title Sheets for PUBLIC ROADS, volumes 24, 25, and 26.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1952

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$14,2[illegible]	$26,072	$13,454	470.7	$8,142	$4,072	200.2	$26,293	$13,420	299.5	$60,507	$30,946	970.4
Arizona	[illegible],307	2,160	1,490	57.5	795	544	16.9	6,556	3,881	73.6	9,511	5,915	148.0
Arkansas	1[illegible],434	11,225	5,938	334.5	1,133	583	57.3	13,538	6,939	296.3	25,896	13,460	688.1
California	20,549	22,824	11,469	110.6	8,763	4,517	53.2	95,319	46,063	211.4	126,906	62,049	375.2
Colorado	9,588	6,568	3,622	107.8	2,051	1,095	51.9	8,914	4,358	113.7	17,533	9,075	273.4
Connecticut	8,180	4,500	2,300	20.3	697	348	2.4	10,886	5,557	17.0	16,083	8,205	39.7
Delaware	3,471	679	340	.8	1,061	530	3.3	6,703	3,389	31.9	8,443	4,259	36.0
Florida	9,830	13,375	6,719	177.9	11,226	5,821	131.8	13,804	6,983	230.1	38,405	19,523	539.8
Georgia	15,577	11,958	6,202	300.6	8,070	4,048	94.3	33,457	16,020	512.7	53,485	26,270	907.6
Idaho	7,370	10,210	6,148	281.8	1,893	1,190	47.7	7,077	4,556	87.3	19,180	11,894	416.8
Illinois	29,902	32,962	18,083	309.9	19,679	10,172	109.1	53,760	27,876	405.9	106,401	56,131	824.9
Indiana	19,932	30,854	15,820	135.3	8,320	4,170	126.4	22,440	12,254	131.5	61,614	32,244	393.2
Iowa	15,336	10,912	5,850	212.5	718	372	28.7	10,587	5,347	459.5	22,217	11,569	700.7
Kansas	15,311	7,327	3,581	776.4	5,238	2,576	466.1	11,454	6,159	554.2	24,019	12,316	1,796.7
Kentucky	11,200	12,868	7,022	193.9	6,056	3,224	130.3	13,270	6,551	206.2	32,194	16,797	530.4
Louisiana	9,911	14,386	7,191	111.5	4,872	2,422	33.0	22,922	11,006	137.5	42,180	20,619	282.0
Maine	4,445	6,483	3,529	25.3	315	239	.3	11,324	5,317	82.1	18,122	9,085	107.7
Maryland	10,769	7,788	3,581	57.7	1,584	625	15.9	9,448	5,103	35.2	18,820	9,309	108.8
Massachusetts	13,274	4,232	2,266	14.5	1,158	574	3.1	44,270	21,104	35.1	49,660	23,944	52.7
Michigan	19,307	20,492	10,414	361.4	5,943	2,953	64.3	56,056	24,154	242.7	82,491	37,521	668.4
Minnesota	16,233	8,538	4,764	962.5	1,557	888	69.6	9,876	5,549	235.6	19,971	11,201	1,267.7
Mississippi	10,930	13,174	6,693	470.9	3,185	1,579	103.5	15,444	8,237	404.9	31,803	16,509	979.3
Missouri	17,358	33,501	17,139	895.3	7,050	3,525	103.0	36,986	19,399	388.6	77,537	40,063	1,386.9
Montana	14,688	8,400	5,058	252.9	1,306	785	16.9	13,841	8,312	231.9	23,547	14,155	501.7
Nebraska	19,266	8,771	4,752	418.4	3,934	2,535	44.2	9,868	4,889	264.2	22,573	12,176	726.8
Nevada	6,621	6,176	4,868	191.6	725	606	2.5	4,007	3,000	120.7	10,908	8,474	314.8
New Hampshire	3,381	4,478	2,239	25.8	553	273	4.4	4,373	2,316	25.7	9,404	4,828	55.9
New Jersey	6,590	12,074	5,891	40.4	14,775	6,855	5.3	28,534	14,098	32.2	55,383	26,844	77.9
New Mexico	8,376	1,682	1,076	40.6	1,858	1,189	54.4	6,728	4,304	172.7	10,268	6,569	267.7
New York	46,288	84,719	44,622	148.6	25,904	12,283	36.5	121,244	55,598	401.0	231,867	112,503	586.1
North Carolina	15,130	18,272	8,913	292.7	6,590	3,023	109.4	25,534	12,352	426.7	50,396	24,288	828.8
North Dakota	8,779	4,601	2,433	697.8	738	369	102.1	5,833	2,955	517.8	11,172	5,757	1,317.7
Ohio	27,150	16,916	8,813	101.5	6,902	3,751	20.9	84,113	41,728	142.7	107,931	54,292	265.1
Oklahoma	14,413	10,056	5,623	149.2	5,581	2,800	130.4	20,176	10,566	245.5	35,813	18,989	525.1
Oregon	5,852	6,340	3,725	65.4	1,538	941	10.5	11,348	6,417	161.5	19,226	11,083	237.4
Pennsylvania	33,399	14,288	7,137	29.9	13,135	6,560	49.6	80,774	40,170	167.0	108,197	53,867	246.5
Rhode Island	3,354	3,793	1,897	29.5	769	384	2.7	18,258	9,507	26.7	22,820	11,788	58.9
South Carolina	8,733	10,234	5,487	155.4	2,279	1,215	125.3	15,287	7,843	302.7	27,800	14,545	583.4
South Dakota	7,826	4,168	2,374	365.1	2,899	1,646	142.7	6,208	3,876	362.0	13,275	7,896	869.8
Tennessee	14,224	8,038	3,992	320.8	7,957	3,984	116.3	28,832	13,174	291.1	44,827	21,150	728.2
Texas	31,729	7,851	3,951	160.0	15,672	8,863	376.4	54,919	29,124	925.8	78,442	41,938	1,462.2
Utah	3,898	4,741	3,640	68.7	2,889	2,102	79.8	9,788	7,392	155.1	17,418	13,134	303.6
Vermont	3,170	2,863	1,616	28.0	1,007	499	7.6	6,075	3,035	40.9	9,945	5,150	76.5
Virginia	12,488	10,135	5,030	128.9	5,318	2,609	110.5	25,113	11,982	200.0	40,566	19,621	439.4
Washington	10,709	6,877	3,776	97.2	2,082	1,114	37.4	7,844	4,463	55.6	16,803	9,353	190.2
West Virginia	6,113	6,467	3,267	33.6	6,771	3,464	41.6	16,892	8,427	135.6	30,130	15,158	210.8
Wisconsin	16,554	8,849	5,178	142.2	3,507	1,884	48.4	30,909	15,158	351.0	43,265	22,220	541.6
Wyoming	5,818	615	423	18.6	795	538	18.6	5,362	3,573	96.0	6,772	4,534	133.2
Hawaii	2,850	2,457	1,206	5.4	1,800	899	7.3	13,483	6,098	41.9	17,740	8,203	54.6
District of Columbia	4,330	13,014	5,686	1.2				4,895	2,444	.7	17,909	8,130	1.9
Puerto Rico	5,891	9,499	4,640	67.1	1,034	471	3.2	14,195	6,790	53.2	24,728	11,901	123.5
TOTAL	648,160	609,462	320,928	10,466.1	247,824	127,709	3,617.2	1,214,817	608,813	11,140.4	2,072,103	1,057,450	25,223.7

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VOL. 27, NO. 7

APRIL 1953

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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In this issue: Road-user and property taxes on selected vehicles
(New Jersey State Route 1 in Jersey City)

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A JOURNAL OF HIGHWAY RESEARCH

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
THOMAS H. MacDONALD, Commissioner

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Road-User and Property Taxes on Selected Motor Vehicles, 1953

BY THE RESEARCH REPORTS BRANCH
BUREAU OF PUBLIC ROADS

Reported by EDWIN M. COPE, Chief
Highway Statistics Section,
and RICHARD W. MEADOWS,
Transportation Economist

Faced with the need to find additional funds for the necessary acceleration of the highway improvement program, administrators, economists, and engineers are studying all available and prospective sources of highway revenue. Highway finance theories all seek a common aim—the equitable levying of costs, through taxation or other means, on those who use and benefit from our roads and streets.

The need for basic factual information in such studies is self-evident. In this article are presented data, by States, on the State road-user imposts—registration fees and motor-fuel taxes—which furnish the bulk of the current income for highways. Since direct property taxes levied on motor vehicles in many States are an important part of the cost of vehicle ownership and operation, data on these taxes are included even though they are not collected as highway revenue. For purposes of comparison, information is reported separately for some of the commercial vehicles according to their use in private operation, farm service, or contract hauling.

The data are reported for eleven typical vehicles, operating under uniform assumed conditions, with tax rates effective on January 1, 1953. Comparisons can thus be made of the taxes paid for similar vehicles in different States, for different types of vehicles in the same State, and for a particular vehicle in different types of operation.

The ranges found are extensive, varying in total amount paid annually from $20 to $4,475. The national average for a farm pickup truck is only $44 a year (less than for a light passenger car), while for a six-axle diesel combination in contract operation it is more than $3,000. Annual taxes on a stake truck average $75 if in farm service, $140 in private use, and $230 in contract hauling operation. A three-axle combination in private use, on the average, pays $256 in registration fees, $422 in motor-fuel tax, and $164 in property tax.

TODAY there are 44 million automobiles and more than 9 million trucks and busses on the highways—53 million vehicles, a total undreamed of 10 years ago. Partly as a result of the great increases in the number of vehicles and partly because of enforced neglect of our highways during the war years and difficulties encountered since, public authorities, legislatures, and highway-user groups are engaged in a desperate effort to accelerate the highway improvement program. The most pressing problem faced is that of finding the necessary funds. A considerable amount of study has been given to highway finance theories—the increment theory, the theory of ability to pay, the cost and the value of the service, and the portion that should be attributed to defense. As a preliminary to any work in tax theory, there is need for measurement of the amounts already being collected under existing tax rates. In the search for revenue, full consideration must be given to the burden of all direct taxes, including property taxes, on the vehicle owner.

The principal current State road-user revenues are derived from the gasoline tax and from registration fees on motor vehicles, but in some States a relatively low gasoline tax is accompanied by above-average registration fees, while in others, higher gasoline taxes are accompanied by low registration fees. In Alabama an automobile can be registered for $3.00 and the gasoline tax rate is 6 cents; in New York State it costs five times as much to register the same automobile but the gasoline tax is only 4 cents. The result is that the road-user taxes are about the same on that automobile in the two States.

The purpose of this study is to provide data that make it possible to compare the total direct State and personal-property taxes on the road user, as well as the yields from the individual levies. It supplies the needed tool of direct measurement of existing taxes, so that comparisons can be made between vehicles, between States, and between different services. This is accomplished through presentation of the amounts of State road-user taxes and all direct personal-property taxes that would be paid in each State (except Oregon) during one full year, at rates in effect January 1, 1953, for each of a group of vehicles that represent significant points in the tax range. The study is similar in many respects to the

NOS 1 & 2
PASSENGER CAR

NO. 3 - PICKUP

NO 4 - STAKE

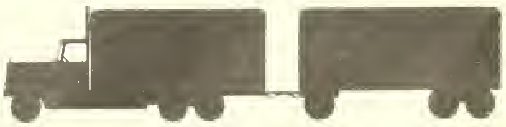
NO 5 - VAN

NO 6 - 3-AXLE DUMP

NO. 7 - 3-AXLE TRACTOR-SEMITRAILER

NOS 8 & 9 - 4-AXLE TRACTOR-SEMITRAILER

NO 10 - 5-AXLE TRACTOR-SEMITRAILER


NO 11 - 6-AXLE TRUCK - FULL TRAILER

one presented for 1950 in PUBLIC ROADS.[1] The 1953 information is summarized or illustrated in tables, bar charts, and maps. The presentations are intended to be self-explanatory and not every one is discussed in detail in the text.

The absence of data for Oregon is a regrettable deficiency. At the time the study was being prepared, the Oregon weight-distance tax was subject to popular referendum in the November 4, 1952, general election, and even if approved would have been invalidated by the approval of a constitutional amendment appearing on the same ballot.[2] Since the imposition of the Oregon weight-distance law was contingent upon both of these pending actions, and could have been invalidated by the outcome of either, data for the State were not included. Consequently, where the phrase "all States" is used, it includes the District of Columbia (in order to avoid the necessity of naming it separately) and all States except Oregon. The absence of Oregon data obviously affects the ranking of States according to total taxes that would be paid on specific vehicles under the prescribed conditions. If it had been possible to include data for Oregon, the ranking of taxes for most of the heavier vehicles would have been affected for all or nearly all States, particularly for vehicles in private, not-for-hire operation.

The Typical Vehicles

The bases for registering motor vehicles vary considerably among the States. As shown in table 1 (p. 129), passenger cars are registered in some States on a flat fee basis, with no regard for other factors. In Mississippi, at the other extreme, a flat fee rate is also in effect but is compounded by consideration of horsepower, gross weight, and vehicle age. The variations in registration bases are shown for passenger cars in figure 1, and for trucks in figure 2 (p. 130). State gasoline tax rates are represented in figure 3 (p. 130).

Eleven vehicles that are reasonably representative of the types and sizes that comprise the vehicle population were selected for the study—two passenger cars, four single-unit trucks, and five combinations. Their relative sizes and axle arrangements are shown in silhouette on page 127. Brief descriptions of the vehicles follow.

PASSENGER CARS:

No. 1.—A light-weight club coupe.

No. 2.—A medium-weight sedan.

SINGLE-UNIT TRUCKS:

No. 3.—A pickup truck registered for 4,700 pounds gross vehicle weight (commonly called a "½-ton" truck).

No. 4.—A stake truck registered for 12,500 pounds gross vehicle weight (commonly called a "1½-ton" truck).

No. 5.—A van registered for 19,000 pounds gross vehicle weight (commonly called a "2¾-ton" truck).

No. 6.—A tandem-axle dump truck registered for 40,000 pounds gross vehicle weight, of the type used in hauling coal, building materials, etc.

COMBINATIONS:

No. 7.—A three-axle tractor-semitrailer registered for 40,000 pounds gross vehicle weight, chosen so that it falls within the maximum length and weight limits of all States.

No. 8.—A four-axle tractor-semitrailer (tandem axles on the semitrailer), gasoline powered, and registered for 50,000 pounds gross vehicle weight (No. 9 is the same vehicle, but with diesel power).

No. 9.—A four-axle tractor-semitrailer (tandem axles on the semitrailer), diesel powered, and registered for 50,000 pounds gross vehicle weight (No. 8 is the same vehicle, but with gasoline power).

No. 10.—A five-axle tractor-semitrailer (tandem axles on both units), registered for 64,000 pounds gross vehicle weight.

No. 11.—A six-axle truck and full trailer combination (tandem axles on both units), registered for 72,000 pounds gross vehicle weight.

The 40,000-pound combination (No. 7) would be permitted to register and operate in all States, but the 40,000-pound tandem-axle single-unit truck (No. 6) would not. Washington has a flat limitation of 36,000 pounds on a truck with three axles, and Mississippi has a limit of 37,650 pounds for such vehicles. Because of axle spacing and axle-load limits, the 40,000-pound single-unit truck is on the borderline of legality in some of the States with respect to registration and operation, but it has been included in order to show that point in the tax schedules. Although the 40,000-pound single-unit truck is found chiefly in specialized operations, the 40,000-pound three-axle combination is in almost universal use and is undoubtedly the most commonly found heavy unit.

Three States have size and weight limitations that prohibit the use of tractor-semitrailer combinations as heavy as 50,000 pounds, but all others permit them. The limit in Kentucky and Tennessee is 42,000 pounds for any combination and Pennsylvania permits a maximum of 45,000 pounds for tractor-semitrailers and 62,000 pounds for other combinations.

In order to present a comparison of the taxes on vehicles similar in most respects except type of fuel used, two 50,000-pound four-axle combinations are included in the study, one with a gasoline engine, the other with diesel power. The assumption made that the gasoline combination operates 4 miles to the gallon and that the fuel consumption rate of the diesel combination is 6 miles per gallon is arbitrary, but believed to be reasonable. It is recognized that the difference in the miles-per-gallon rates of two individual vehicles might fall substantially short of, or could exceed, the assumed 50-percent differential. (A 50-percent difference in consumption may also be expressed as a one-third saving in fuel.) It is of interest that New York also assumed a 50-percent differential in recently increasing the tax on diesel fuel from 4 cents to 6 cents per gallon.

The 64,000-pound combination (No. 10) selected is a five-axle, diesel-powered, tractor-semitrailer combination, 49 feet long, that can be operated in the Western States and a few Eastern States that are not contiguous, as shown in figure 5 (p. 131). The 64,000-pound combination in the 1950 study had a gasoline engine, but the growing popularity of diesel power for busses and heavy trucks dictated the selection of diesel power for both the 64,000-pound and the 72,000-pound combinations in the present study. The 72,000-pound truck and full trailer combination (No. 11) is permitted in Ohio, Louisiana, and all of the Western States. Ohio is the only State that permits vehicle No. 11 but prohibits vehicle No. 10 (see fig. 5). This is because of differences in the State's length limitations for the two types of combinations.

As shown in figure 6, tractor-semitrailer combinations are registered as single units in all 6 of the New England States, in 12 of the States in the Mississippi and Ohio River valleys, and in Virginia. In the remainder of the States, tractor-trucks and semitrailers are registered as separate units. Combinations are considered to be registered as single units where the fee for either the tractor or the semitrailer is based on the gross weight or capacity of the combination, even though in most of these States a relatively small fee is also levied on the other unit of the combination.

In some States the registration fee schedule is so established that from a tax standpoint it is more economical to operate a combination than a single-unit truck at a given gross vehicle weight. In Montana, for example, registration of a 40,000-pound single-unit truck costs $300.00, but the tractor and semitrailer components of a 40,000-pound combination, registered separately, cost only $92.50 in total.

The inclusion or exclusion of data for a given vehicle in each State was determined solely on the basis of whether the vehicle as specified would be permitted to register and operate. Numerous instances were found in which the vehicle or combination might be registered, but could not legally be operated with a full load. In New York, for instance, vehicle No. 10 could be registered

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[1] *State road-user and personal-property taxes on selected motor vehicles, 1950*, by R. W. Meadows and S. F. Bielak. PUBLIC ROADS, vol. 26, No. 2, June 1950.

[2] In the election, Oregon voters approved the increased weight-distance taxes enacted into law by the 1951 State legislature.

Table 1.—Basis for registration of passenger cars

FLAT FEE: Alabama Arizona California Idaho Kentucky Louisiana Nevada Ohio Oregon Pennsylvania Utah Virginia Washington Wisconsin Wyoming Flat Fee and Age: Vermont	**EMPTY WEIGHT** Weight Groups: Connecticut Delaware Florida Maryland Montana North Carolina Tennessee Texas District of Columbia Weight Groups and Horsepower: Indiana Weight Groups and Age: North Dakota South Dakota	100-Pound Intervals: Colorado Michigan West Virginia 100-Pound Intervals and Age: New Mexico 100-Pound Intervals, Value and Age: Iowa 500-Pound Intervals: Georgia	**SHIPPING WEIGHT** Weight Groups: Nebraska Weight Groups and Age: Minnesota 100-Pound Intervals: New York 500-Pound Intervals: South Carolina
HORSEPOWER GROUPS: Illinois Maine Massachusetts Missouri New Jersey	**GROSS WEIGHT** Weight Groups: New Hampshire Rhode Island Weight Groups and Horsepower: Arkansas	100-Pound Intervals: Kansas	Factory Delivered Price and Age: Oklahoma Flat Fee, Horsepower, Gross Weight, and Age: Mississippi

Table 2.—Data used in computing State road-user taxes and property taxes on selected vehicles, 1953 registration year

Factors affecting taxation	Passenger cars		Single-unit trucks				Tractor-semitrailer combinations								Truck and full trailer	
	No. 1: Light	No. 2: Medium	No. 3: Pickup	No. 4: Stake	No. 5: Van	No. 6: Dump	No. 7		No. 8 (gasoline)		No. 9 (diesel)		No. 10		No. 11	
							Tractor-truck	Semi-trailer	Tractor-truck	Semitrailer	Tractor-truck	Semitrailer	Tractor-truck	Semitrailer	Van truck	Full trailer
Body type	Club coupe, 6-pass.	Four-door, 6-pass.	Pickup	Stake	Van	Dump	Tractor	28-ft. van	Tractor	32-ft. van	Tractor	32-ft. van	Tractor	35-ft. van	Van	24-ft. van
Price, f.o.b. factory (1951)	$1,575	$2.148	$1,250	$1,784	$4,859	$11,270	$6,102	$4,400	$6,350	$6,862	$10,327	$6.931	$16,987	$7.000	$18,399	$7,500
Horsepower:																
N.A.C.C. (S.A.E.)	25.35	32.51	26.10	29.40	33.75	40.80	40.84		38.40		48.84		63.00		63.00	
Maximum brake	97	124	95	92	137	147	155		135		150		165		200	
Weight:																
Manufacturer's rated capacity ... tons			½	1½	2¾											
Chassis weight ... pounds				4,135	6.425	12,675									17,800	
Empty weight ... do	3,059	3,755	3,220	5,320	8,625	18,862	8,825	7,320	8,400	8.700	9,550	8,990	16,250	8,940	20,300	8,500
Load capacity ... do			1,480	7,180	10,375	21,138	13,175	10,680	15,600	17,300	15,450	16,010	19,750	19,060	15,700	27,500
Gross vehicle weight ... do	3,959	4,655	4,700	12,500	19,000	40,000	22,000	18,000	24,000	26,000	25,000	25,000	36,000	28,000	36,000	36,000
Gross weight of combination ... do							40,000		50,000		50,000		64,000		72,000	
Length:																
Wheelbase ... inches			114	161	172	159–48½	141		136		134		169–50		210–52	176–52
Distance between tandem axles ... do						48½				48½		48½	50	48½	52	52
Length of combination ... feet							39		42		42		49		59	
Tires:																
Size			6.50 x 16	7.50 x 20	8.25 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 22	10.00 x 22
Ply rating			6	8	10	12	12	12	12	12	12	12	14	14	12	12
Front arrangement			Single	Single	Single	Single	Single		Single		Single		Single		Single	Dual
Rear arrangement			Single	Dual	Dual	Dual-tandem	Dual	Dual	Dual	Dual-tandem	Dual	Dual-tandem	Dual-tandem	Dual-tandem	Dual-tandem	Dual-tandem
Total annual travel:																
Farm service ... miles			6,000	5,500												
Private operation ... do	9,500	10,500	9,000	12,000	15,000	30,000	40,000		65,000		65,000		80,000		90,000	
Contract carrier ... do				20,000	25,000		40,000		65,000		65 000		80,000		90,000	
Average miles per gallon:																
Farm service			15.5	10.0												
Private operation	16.5	14.5	15.0	9.0	6.5	4.0	5.0		4.0		6.0		4.7		4.3	
Contract carrier				9.0	6.5		5.0		4.0		6.0		4.7		4.3	
Annual motor-fuel consumption:																
Farm service ... gallons			387	550												
Private operation ... do	576	724	600	1,333	2,307	7.500	8,000		16,250		10,833		17,021		20,930	
Contract carrier ... do				2,222	3,846		8,000		16,250		10,833		17,021		20,930	
Total revenue ton-miles (average load)				40,000	71,000		285,000		695,000		665,000		1,085,000		1,400,000	
Average gross weight ... pounds				9,270	14,325		30,458		38,485		38,989		52,357		59,900	
Gross annual earnings				$8,000	$11,500		$23,000		$52,000		$50,000		$78,000		$96,000	

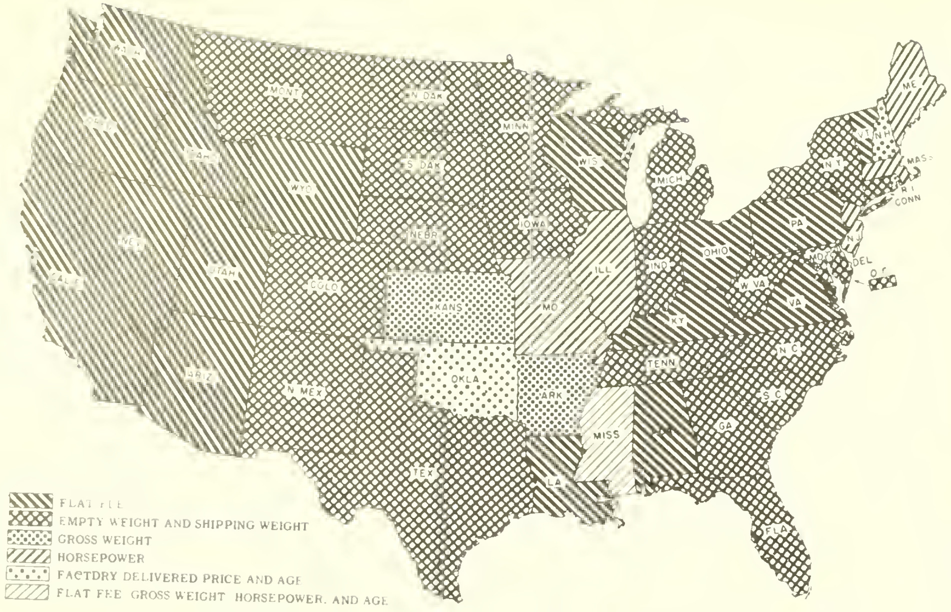

FLAT FEE
EMPTY WEIGHT AND SHIPPING WEIGHT
GROSS WEIGHT
HORSEPOWER
FACTORY DELIVERED PRICE AND AGE
FLAT FEE GROSS WEIGHT HORSEPOWER, AND AGE

Figure 1.—Passenger-car registration fee basis.

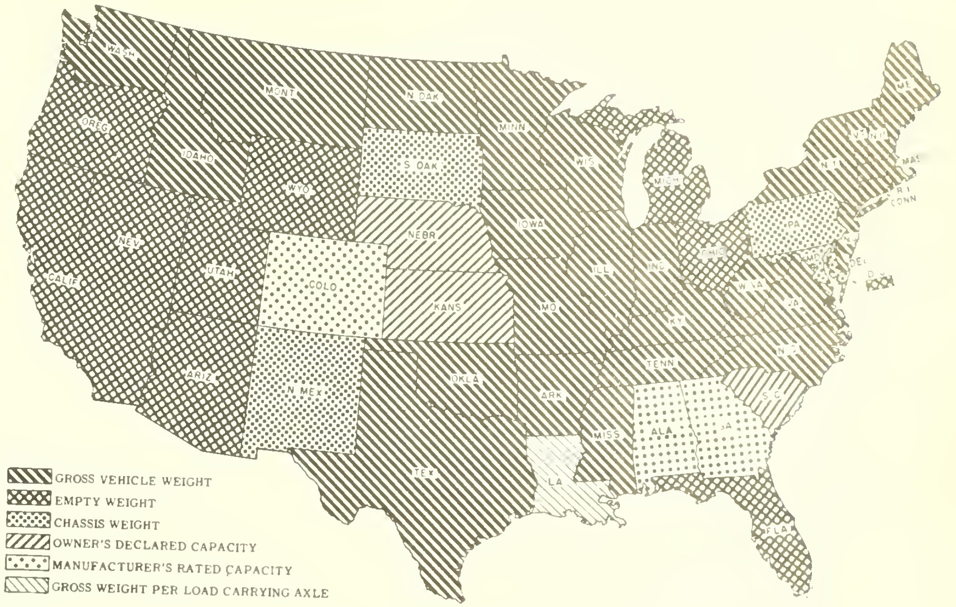

GROSS VEHICLE WEIGHT
EMPTY WEIGHT
CHASSIS WEIGHT
OWNER'S DECLARED CAPACITY
MANUFACTURER'S RATED CAPACITY
GROSS WEIGHT PER LOAD CARRYING AXLE

Figure 2.—Truck registration fee basis.

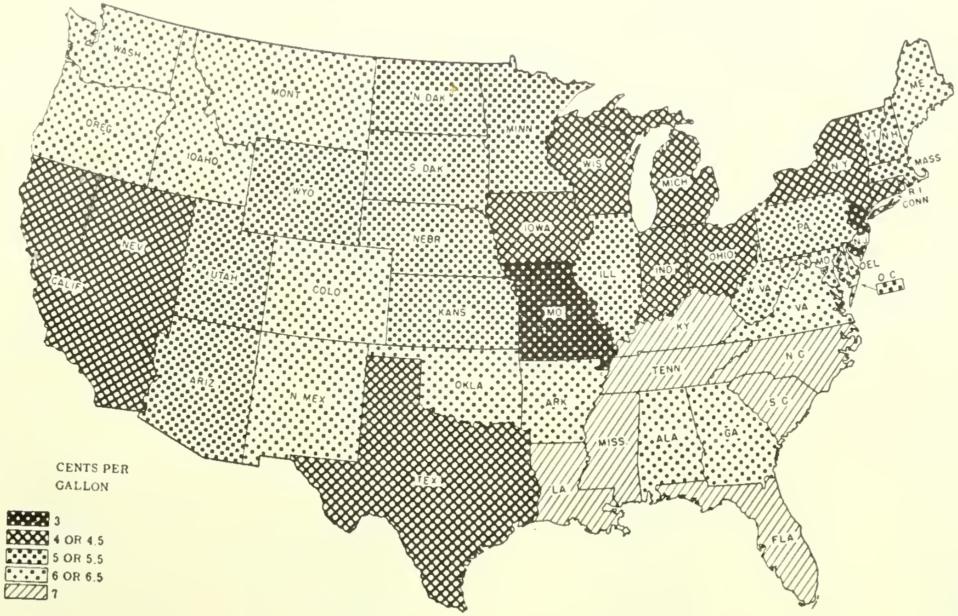


Figure 3.—State gasoline tax rates.

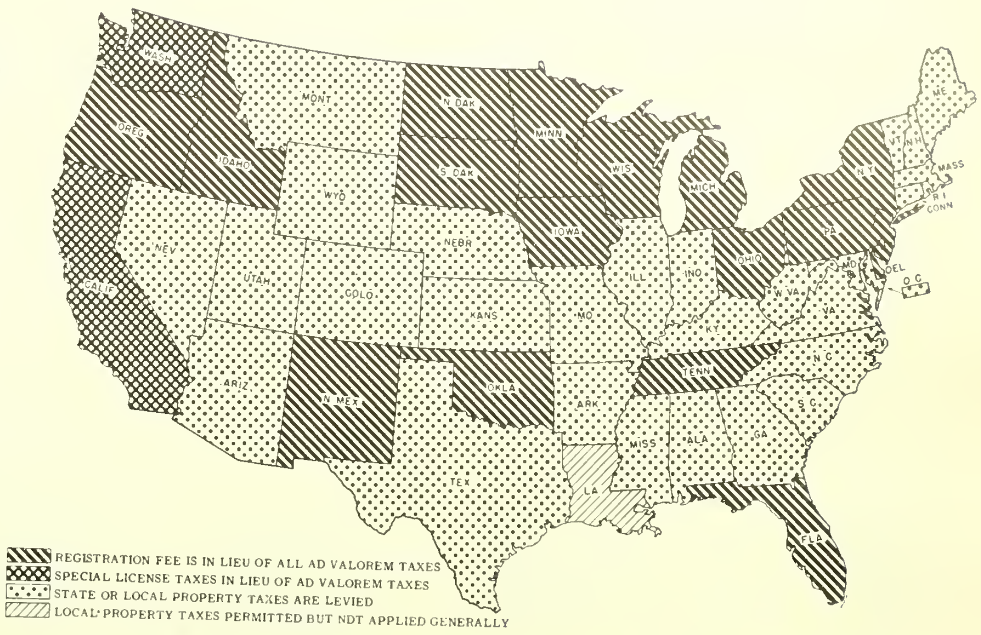

REGISTRATION FEE IS IN LIEU OF ALL AD VALOREM TAXES
SPECIAL LICENSE TAXES IN LIEU OF AD VALOREM TAXES
STATE OR LOCAL PROPERTY TAXES ARE LEVIED
LOCAL PROPERTY TAXES PERMITTED BUT NOT APPLIED GENERALLY

Figure 4.—Application of personal-property taxes to motor vehicles.

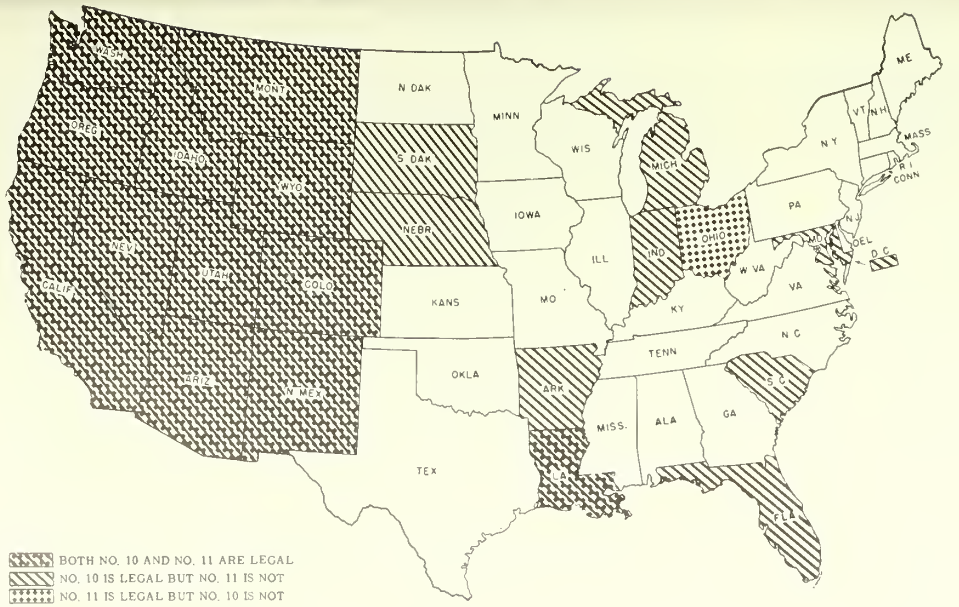


Figure 5.—Legality of 64,000-pound tractor-semitrailer (No. 10) and 72,000-pound truck and full trailer (No. 11).

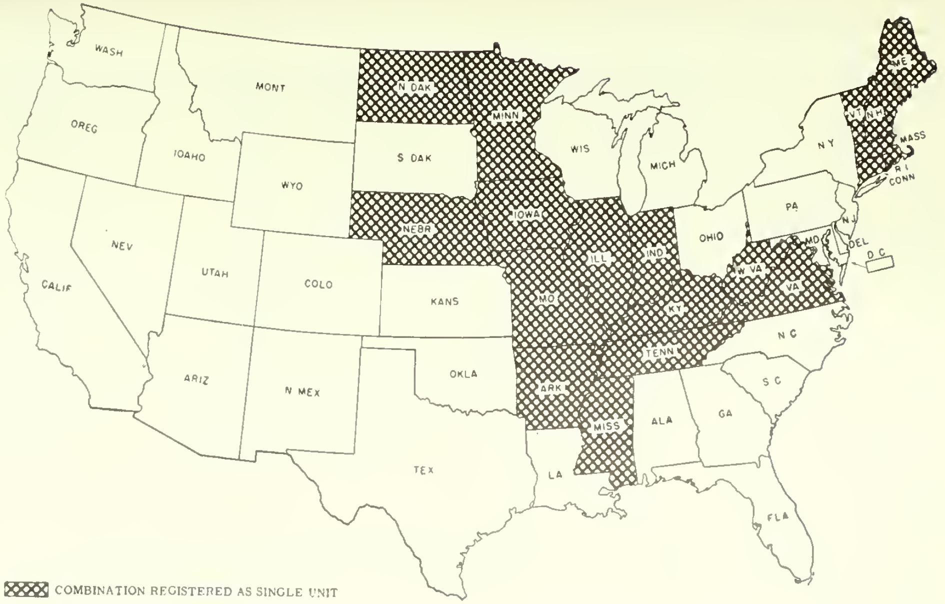


Figure 6.—Registration of tractor-semitrailer as a single unit.

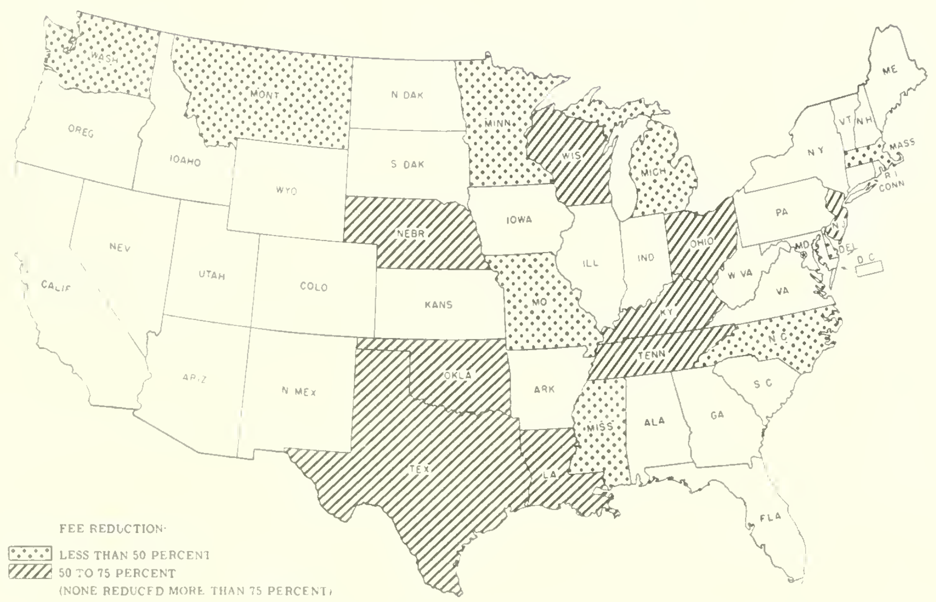


Figure 7.—Reduced registration fees for farm truck (No. 3, pickup truck).

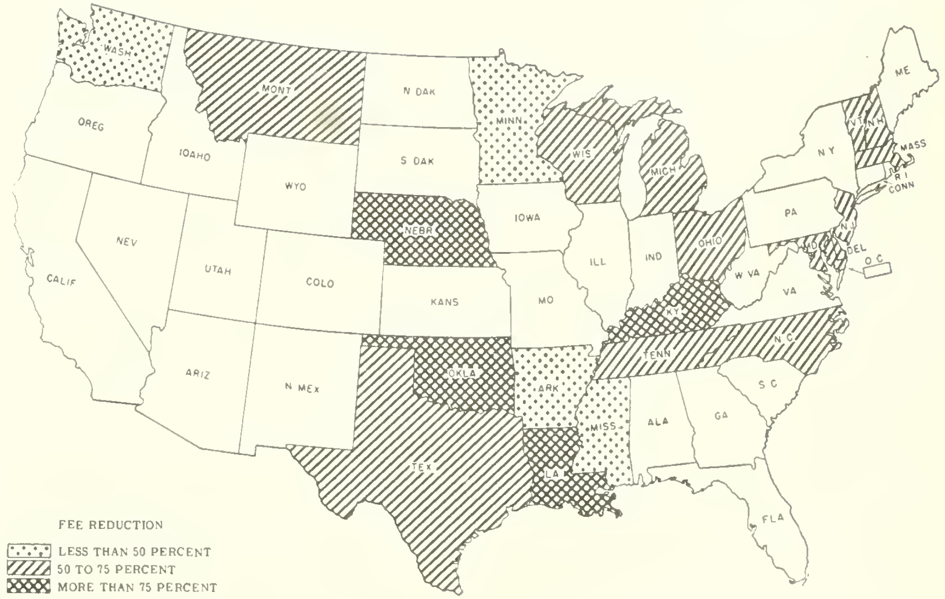


Figure 8.—Reduced registration fees for farm truck (No. 4, stake truck).

but would be permitted to operate at a maximum gross weight of approximately 62,000 pounds. Consequently, it was omitted from the study even though it was eligible to be registered for a slightly smaller gross weight. Several similar situations existed with respect to the heavy vehicles and all were omitted unless they could be legally operated at the gross vehicle weights and sizes specified. Minor adjustments of axle spacing were assumed in a few cases, however, in order to include vehicles that otherwise were within the legal limits.

Specifications and Use Factors

In order to obtain the tax data on a uniform basis in all of the States, a detailed outline of the specifications of each of the 11 vehicles selected was submitted to the State authorities in the latter part of 1952 for determination of the specific fees and taxes that would be payable during a full year under laws existing January 1, 1953 (it was the uncertainty on this point that necessitated the exclusion of Oregon). Table 2 (p. 129) gives these specifications, with the assumed annual mileage traveled, motor fuel consumed, and other factors needed to compute the various taxes. The taxes to be reported were the State road-user taxes—on motor fuel, on vehicle registration, other annual vehicle fees closely related to registration, and the applicable motor-carrier taxes—and all direct personal-property taxes levied on motor vehicles at all levels of government. The property taxes for 1953 were to be estimated by the State authorities on the basis of 1952 property valuations and tax rates.

The same care that was taken in the selection of vehicles was taken in the selection of use factors. "Average" values are not available, however, for the mileages, fuel consumption rates, and earnings. It was therefore necessary to assign these values arbitrarily, but sincere effort was made to select amounts that are reasonable. The primary objective was to supply specific factors to which rates could be applied uniformly in all States, and ones that would reflect these rates in a way that makes possible valid comparisons between States, vehicles, and services.

In order to avoid the complex situations that would have been encountered in computing taxes on vehicles in interstate operation, intrastate use was specified. Without this stipulation, the study would have been impractical. Obviously, however, it is almost unthinkable that large combinations would travel entirely in one State for a whole year—particularly in such States as Rhode Island, Delaware, or for that matter, almost any of the smaller States; and it is improbable that their travel would be restricted to just one of the larger States. However, the stipulation of intrastate travel is permissible, even indispensable, when it is remembered that the purpose of the study is to compare tax rates, rather than to select the most typical vehicles or the most probable amount and kind of service, or the most probable earnings.

It was also specified that the vehicles had been operated in the same State since purchased new in 1951. This made 1953 the third year of registration, thus excluding from the study such nonrecurring taxes as those on sales and titling, and removing the vehicles from the top property-tax brackets.

Another large group of taxes omitted are those imposed by the Federal Government on gasoline, oil, vehicles, and accessories and parts, which yielded over $2 billion in 1952. These are excise taxes, identical in nature with those levied on tobacco, cosmetics, etc. Their exclusion has no effect on the comparisons between States of the data presented in this article, since the levies are uniform throughout the nation.

The data received from the States are presented in tables 4–12. In all of these tables the States are arranged in groupings so as to permit ready comparison of data for each State with those of its neighbors. The national averages and extremes are shown in table 3. The averages are simple unweighted averages of the totals for all States where the vehicles are permitted. In the case of property taxes they are averages reflecting only the States where such taxes are levied. Figures 10–21 portray the data graphically, permitting quick comparisons of the several elements reported (pp. 143-153).

Property Taxes

In most cases the property taxes on motor vehicles have little or no direct relation to the use of highways, and the revenues from them are not available for highways. They are, however, so closely associated with registration fees in their application and their relation to costs of operation, and comprise such a large portion of the total

Table 3.—Average, low, and high road-user and property taxes on selected motor vehicles

Vehicle and service	Average fee for all States [1]					Lowest fee					Highest fee				
	Registration fee, etc.	Motor-fuel tax	Total road-user taxes	Property tax	Total	Registration fee, etc.	Motor-fuel tax [1]	Total road-user taxes	Property tax [1]	Total	Registration fee, etc.	Motor-fuel tax	Total road-user taxes	Property tax	Total
Passenger car:															
Light-weight (No. 1)	$11.95	$30.36	$42.31	$28.41	$58.88	$3.00	$17.28	$28.28	($5.00)	$28.28	$29.26	$40.32	$66.70	$54.32	$88.12
Medium-weight (No. 2)	14.67	38.16	52.83	38.45	75.26	3.00	21.72	32.72	(8.00)	38.96	37.26	50.68	84.82	74.51	115.71
Pickup (No. 3):															
Farm	13 79	20.40	34.19	16.25	43.67	2.50	11.61	20.48	(5.00)	20.48	32 00	27.09	51.34	34.58	64.35
Private	16.97	31.63	48.60	23.86	62.52	2.50	18.00	32.00	(5.00)	44.00	34.80	42.00	68.00	55.06	94.31
Stake truck (No. 4):															
Farm	32.80	28.99	61.79	22.31	74.80	4.00	16.50	31.50	(5.17)	44.50	86.00	38.50	113.50	51.05	127.57
Private	51.07	70.26	121.33	32.96	140.55	10.00	39.99	81.65	(15.00)	100.99	132.50	93.31	199.15	77.50	199.15
Contract	94.16	117.12	211.28	32.96	230.51	25.00	66.66	127.66	(15.00)	127.66	289.00	155.54	401.54	77.50	401.54
Van (No. 5):															
Private	104.65	121.60	226.25	75.92	270.53	26.00	69.21	142.53	(18.25)	160.21	285.00	161.49	400.35	156.16	434.88
Contract	170.12	202.72	372.84	75.92	417.13	59.00	115.38	206.38	(18.25)	206.38	497.50	269.22	766.72	156.16	766.72
Dump (No. 6):															
Private	295.20	390.49	685.69	182.66	792.90	50.00	225.00	427.00	(40.18)	466.00	877.50	525.00	1,252.50	471.95	1,252.50
Three-axle combination (No. 7):															
Private	255.83	421.67	677.50	164.37	773.38	46.00	240.00	442.00	(45.00)	442.00	640.00	560.00	1,040.00	371.32	1,122.37
Contract	399.27	421.67	820.94	164.37	916.82	115.00	240.00	442.00	(45.00)	442.00	915.00	560.00	1,275.00	371.32	1,284.45
Four-axle combination, gasoline (No. 8):															
Private	342.84	845.00	1,187.84	197.79	1,396.52	66.00	487.50	749.50	(45.00)	749.50	961.50	1,137.50	1,637.50	378.13	1,767.00
Contract	588.37	845.00	1,433.37	197.79	1,552.04	130.00	487.50	749.50	(45.00)	749.50	1,824.00	1,137.50	2,555.25	378.13	2,589.54
Four-axle combination, diesel (No. 9):															
Private	415.85	579.05	956.30	259.51	1,112.04	66.00	(324.99)	543.32	(45.00)	586.99	1,524.90	866.64	1,617.98	603.82	1,617.98
Contract	657.53	579.05	1,197.98	259.51	1,353.68	135.00	(324.99)	586.99	(45.00)	586.99	1,816.67	866.64	2,304.16	603.82	2,304.16
Five-axle combination, diesel (No. 10):															
Private	605 69	955.07	1,465.25	413.21	1,713.18	101.00	(680.84)	860.84	(45.00)	1,031.05	2,926.40	1,191.47	2,926.40	777.33	2,926.40
Contract	1,148 47	955.07	2,008.03	413.21	2,255.96	135.00	(680.84)	986.05	(45.00)	1,031.05	2,926.40	1,191.47	3,581.29	777.33	3,581.29
Six-axle combination, diesel (No. 11):															
Private	736 19	1,132.12	1,773.97	453.64	2,000.79	108.50	(837.20)	1,057.20	(310.79)	1,367.99	3,427.20	1,465.10	3,427.20	741.23	3.427.20
Contract	1,844.07	1,132.12	2,881.85	453.64	3,108.67	445.00	(837.20)	1,491.50	(310.79)	1,560.95	3,427.20	1,465.10	4,320.18	741.23	4,475.09

[1] In 20 States, personal-property taxes are not imposed, and in 3 States diesel fuel is not taxed directly The average fees given are for the States where the respective taxes are collected. The fees in parentheses are the lowest for the same States.

Table 4.—Road-user and personal-property taxes on selected passenger vehicles (Nos. 1 and 2)

State	Light-weight passenger car (No. 1)							Medium-weight passenger car (No. 2)						
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State	
						Total fees and taxes	Total, excluding property tax						Total fees and taxes	Total, excluding property tax
New England:														
Maine	$14.00	$19.69	....	$34.56	$68.25	14	14	$16.00	$26 85	....	$43.44	$86.29	15	15
New Hampshire	14.00	14.18	....	28.80	56.98	24	22	21.15	19.33	....	36.20	76.68	22	17
Vermont	26.00	....	....	28.80	54.80	28	4	26 00	....	....	36.20	62.20	35	12
Massachusetts	4.50	31.50	....	28.80	64.80	18	42	6.50	42.96	....	36.20	85.66	16	38
Rhode Island	12.00	45.28	....	23.04	80.32	6	35	16.00	54 68	....	28.96	99.64	8	34
Connecticut	7.00	38.61	....	23.04	68.65	13	46	9.00	52.55	....	28.96	90.51	12	46
Middle Atlantic:														
New York	15.50	....	....	23.04	38.54	44	30	19.75	....	....	28.96	48.71	41	26
New Jersey	10.00	....	$1.00	17.28	28.28	48	48	16.50	....	$1.00	21.72	39.22	47	44
Pennsylvania	10.00	....	....	28.80	38.80	43	29	10.00	....	....	36.20	46.20	43	32
Delaware	8.00	....	....	28.80	36 80	45	33	8 00	....	....	36.20	44.20	46	36
Maryland	10.00	5.00	....	28.80	43.80	39	28	15.00	8 00	....	36 20	59.20	38	24
District of Columbia	5.00	22.00	....	28 80	55.80	26	41	8.00	32.00	....	36.20	76.20	23	37
West Virginia	17.60	39.78	....	28.80	86.18	2	16	21.80	53.70	....	36.20	111.70		16
Southeastern:														
Virginia	10.00	26.18	....	34.56	70.74	12	20	10.00	34 32	....	43.44	87.76	14	22
North Carolina	10.00	27.93	....	40.32	78.25	8	11	12.00	37.48	....	50 68	100.16	7	11
South Carolina	5.00	36.00	....	40.32	81.32	4	18	6.00	48.00	....	50.68	104.68	4	18
Georgia	3.50	29 28	....	34.56	67.34	15	32	4.50	40.72	....	43.44	88.66	13	28
Florida	15.00	....	.25	40.32	55.57	27	3	20.00	....	25	50 68	70.93	27	4
Kentucky	4.50	33.25	.50	40.32	78.57	7	17	4.50	45.34	.50	50.68	101 02	6	19
Tennessee	7.50	....	.50	40.32	48.32	37	15	10 00	....	.50	50.68	61.18	37	14
Alabama	3.00	18.09	.75	34.56	56.40	25	31	3.00	26 80	.75	43.44	73.99	25	30
Mississippi	10.37	35.10	....	40.32	85.79	3	9	12.62	48.06	....	50.68	111.36	3	10
East Central:														
Ohio	10.00	....	....	23.04	33.04	47	43	10.00	....	....	28.96	38.96	48	45
Indiana	11.00	46.78	.25	23.04	81.07	5	37	12.00	62.01	.25	28 96	103.22	5	39
Illinois	10.50	• 18.68	....	28.80	57.98	23	25	10.50	23.28	....	36.20	69.98	29	31
Michigan	10.85	....	....	25.92	36.77	46	34	13.30	....	....	32.58	45.88	44	33
Wisconsin	16.00	....	....	23.04	39.04	42	27	16.00	....	....	28.96	44.96	45	35
Minnesota	24.80	....	....	28.80	53.60	30	6	33.30	....	....	36.20	69.50	30	5
Iowa	27.00	....	....	23.04	50.04	35	12	35.00	....	....	28.96	63.96	34	9
Missouri	11.00	17.04	....	17.28	45.32	38	47	11.00	22.05	....	21.72	54.77	39	48
Southwestern:														
Arkansas	13.00	15.19	....	37.44	65.63	17	10	17.00	21.56	....	47.06	85.62	17	8
Louisiana	3.00	....	....	40.32	43.32	40	21	3.00	....	....	50.68	53 68	40	20
Oklahoma	28.76	....	.50	37.44	66.70	16	1	37.26	....	.50	47.06	84.82	18	1
Texas	11.52	17.47	....	23.04	52 03	34	36	18.72	22.33	....	28.96	70.01	28	29
West Central:														
North Dakota	25.00	....	....	28.80	53.80	29	5	35.00	....	....	36.20	71.20	26	3
South Dakota	20.00	....	....	28.80	48.80	36	13	25.00	....	....	36.20	61.20	36	13
Nebraska	5.00	54.32	....	28.80	88.12	1	38	5.00	74.51	....	36 20	115.71	1	40
Kansas	10.35	38.00	....	28.80	77.15	9	26	12.80	50.00	....	36.20	99.00	9	25
Mountain:														
Montana	10.00	28.70	....	34.56	73.26	11	19	10.00	39.30	....	43.44	92.74	11	21
Idaho	5.00	....	....	34.56	39.56	41	24	5.00	....	....	43.44	48.44	42	27
Wyoming	5.00	18.90	....	28.80	52.70	33	40	5.00	25.78	....	36.20	66.98	32	42
Colorado	5.75	18.90	1.00	34.56	60.21	22	23	6.80	25.78	1.00	43.44	77.02	21	23
New Mexico	19.00	....	....	34 56	53.56	31	7	26 00	....	....	43.44	69.44	31	6
Arizona	3.50	28.40	....	28.80	60.70	20	44	3.50	38 80	....	36.20	78.50	20	43
Utah	5.00	28.52	....	28.80	62.32	19	39	5.00	40.30	....	36.20	81.50	19	41
Nevada	5.00	42.79	....	25.92	73.71	10	45	5.00	60.10	....	32.58	97.68	10	47
Pacific:														
Washington	5.00	....	18.25	37.44	60.69	21	2	5.00	....	24.00	47.06	76.06	24	2
Oregon	....	....	....	....	....	....	....	....	....	....	....	....	....	....
California	6.00	....	21.00	25.92	52.92	32	8	6.00	....	27 00	32.58	65.58	33	7

taxes paid on motor vehicles in some States, that their inclusion was necessary in order to obtain equitable comparisons. The registration fees for individual vehicles vary considerably among the States, and property taxes in some instances greatly exceed the registration fees paid. When property taxes and registration fees are considered together, however, the differences among the States are smaller in most instances.

In figure 4 (p. 130) are shown the States in which State or local property taxes are levied and the States in which registration fees are in lieu of ad valorem or property taxes. In this connection it should be noted that the registration fees of quite a few States are based at least in part on the age or valuation of the vehicle. Louisiana permits local property taxes to be levied in addition to the registration fee, but the application of such levies is not general in the State. The relation of property taxes to registration fees and total taxes on motor vehicles is shown in tables 4–12.

It is worth noting that the valuation of motor vehicles for tax purposes is generally conceded to be much closer to market value than is the valuation of other personal property (household furnishings, for instance) subject to the same taxes. It is also probable that motor vehicles constitute a very considerable portion of taxable personal property in many jurisdictions. Therefore, although this study is not directly concerned with personal-property taxes per se, it suggests an area of research that must not be neglected by those interested in the tax burdens on the motor-vehicle owner.

It will be seen in table 4 that the registration fee in Pennsylvania for the light two-door passenger car is $10.00, more than twice as much as the $4.50 shown for Massachusetts. In Pennsylvania, however, the flat registration fee of $10.00, which applies to all automobiles, is in lieu of all other ad valorem taxes, whereas in Massachusetts vehicles are subject to property taxes in addition to registration fees. The addition of the $31.50 property tax to the $4.50 registration fee yields a total of $36.00, which is 3.6 times as much as the Pennsylvania registration fee and much greater than the registration fee alone for any State. North Dakota and Vermont, with registration fees of $25.00 and $26.00, respectively, appear to be exacting greater taxes on the light automobile than other States. Yet in neither of these States is there a property tax on motor vehicles, and when property taxes and registration fees are combined, the total in a great many States exceeds the fees in North Dakota and Vermont. Although Pennsylvania, Massachusetts, North Dakota, and Vermont are used as examples, it may be seen from the tables presented in this study that many other significant comparisons can be made.

Another factor that demonstrates the close relation between registration fees and property taxes is the reduction of the registration fee according to the age (and value) of the vehicle, as may be seen in table 1.

The property taxes that are given in tables 4–12, for all except the farm vehicles, include all taxes levied by the State, county, city, or other local governmental unit in which the vehicle is domiciled, and are the taxes that would be imposed on the vehicles

Table 5.—Road-user and personal-property taxes on a pickup truck, 4,700 pounds gross vehicle weight (No. 3)

State	Farm service							Private operation						
						Rank of State							Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:														
Maine	$15.00	$15.63	...	$23.22	$53.85	9	14	$15.00	$15.63	...	$36.00	$66.63	18	19
New Hampshire	21.15	11.25	...	19.35	51.75	11	5	21.15	11.25	...	30.00	62.40	23	17
Vermont	26.00	...	...	19.35	45.35	20	4	26.00	...	...	30.00	56.00	32	9
Massachusetts	12.00	25.00	...	19.35	56.35	6	33	15.00	25.00	...	30.00	70.00	11	33
Rhode Island	16.50	24.00	...	15.48	55.98	8	30	16.50	34.69	...	24.00	75.19	8	39
Connecticut	14.10	32.18	...	15.48	61.76	3	37	14.10	32.18	...	24.00	70.28	10	42
Middle Atlantic:														
New York	23.50	...	...	15.48	38.98	33	12	23.50	...	...	24.00	47.50	43	29
New Jersey	15.00	...	$1.00	11.61	27.61	47	39	30.00	...	$1.00	18.00	49.00	41	25
Pennsylvania	16.50	...	...	19.35	35.85	36	18	16.50	...	...	30.00	46.50	45	30
Delaware	10.00	...	...	19.35	29.35	46	38	15.00	...	...	30.00	45.00	47	32
Maryland	12.00	5.00	...	19.35	36.35	35	32	12.00	5.00	...	30.00	47.00	44	38
District of Columbia	20.00	17.00	...	19.35	56.35	7	11	20.00	17.00	...	30.00	67.00	17	23
West Virginia	20.00	25.00	...	19.35	64.35	1	10	20.00	31.25	...	30.00	81.25	5	22
Southeastern:														
Virginia	12.00	9.60	...	23.22	44.82	22	22	12.00	20.02	...	36.00	68.02	13	28
North Carolina	10.00	8.74	...	27.09	45.83	18	17	18.80	19.48	...	42.00	80.28	7	4
South Carolina	6.00	11.00	...	27.09	44.09	23	28	6.00	27.00	...	42.00	75.00	9	27
Georgia	2.50	13.28	...	23.22	39.00	32	43	2.50	29.28	...	36.00	67.78	15	41
Florida	24.00	...	.25	27.09	51.34	12	1	24.00	...	.25	42.00	66.25	20	2
Kentucky	4.50	16.00	.50	27.09	48.09	17	29	10.00	32.24	.50	42.00	84.74	3	12
Tennessee	7.50	...	.50	27.09	35.09	40	23	15.00	...	.50	42.00	57.50	31	8
Alabama	15.00	11.34	.50	23.22	50.06	13	13	15.00	18.09	.50	36.00	69.59	12	16
Mississippi	7.00	9.51	...	27.09	43.60	24	25	10.00	29.01	...	42.00	81.01	6	14
East Central:														
Ohio	16.40	...	...	15.48	31.88	43	31	34.80	...	...	24.00	58.80	29	7
Indiana	15.00	31.95	.25	15.48	62.68	2	34	15.00	55.06	.25	24.00	94.31	1	40
Illinois	20.00	10.50	...	19.35	49.85	14	8	20.00	11.25	...	30.00	61.25	25	21
Michigan	16.00	...	...	17.42	33.42	42	27	25.60	...	...	27.00	52.60	36	11
Wisconsin	5.00	...	...	15.48	20.48	48	48	20.00	...	...	24.00	44.00	48	35
Minnesota	20.40	...	...	19.35	39.75	29	7	25.00	...	...	30.00	55.00	35	10
Iowa	25.00	...	...	15.48	40.48	28	6	25.00	...	...	24.00	49.00	40	24
Missouri	15.00	8.29	...	11.61	34.90	41	42	20.00	17.04	...	18.00	55.04	34	43
Southwestern:														
Arkansas	12.00	8.25	...	25.16	45.41	19	16	12.00	12.25	...	39.00	63.25	21	18
Louisiana	3.00	...	...	27.09	30.09	44	36	10.00	...	...	42.00	52.00	37	13
Oklahoma	9.60	...	.50	25.16	35.26	38	20	20.00	...	.50	39.00	59.50	27	5
Texas	9.40	5.17	...	15.48	30.05	45	44	18.80	12.90	...	24.00	55.70	33	37
West Central:														
North Dakota	20.00	...	...	19.35	39.35	30	9	20.00	...	12.00	30.00	62.00	24	3
South Dakota	16.25	...	...	19.35	35.60	37	19	16.25	...	...	30.00	46.25	46	31
Nebraska	4.00	19.82	...	19.35	43.17	25	46	8.00	44.76	...	30.00	82.76	4	44
Kansas	7.50	16.00	...	19.35	42.85	26	40	7.50	25.00	...	30.00	62.50	22	45
Mountain:														
Montana	11.20	19.34	...	23.22	53.76	10	24	16.00	33.25	...	36.00	85.25	2	15
Idaho	12.00	...	...	23.22	35.22	39	21	12.00	...	...	36.00	48.00	42	26
Wyoming	5.00	15.00	...	19.35	39.35	31	45	5.00	15.00	...	30.00	50.00	39	47
Colorado	6.05	15.00	1.00	23.22	45.27	21	35	6.05	15.00	1.00	36.00	58.05	30	36
New Mexico	15.00	...	...	23.22	38.22	34	15	15.00	...	...	36.00	51.00	38	20
Arizona	14.70	22.60	...	19.35	56.65	5	26	14.70	22.60	...	30.00	67.30	16	34
Utah	7.50	14.02	...	19.35	40.87	27	41	7.50	22.32	...	30.00	59.82	26	46
Nevada	5.00	34.58	...	17.42	57.00	4	47	5.00	34.58	...	27.00	66.58	19	48
Pacific:														
Washington	9.75	...	14.50	25.16	49.41	16	3	14.50	...	14.50	39.00	68.00	14	1
Oregon	...	...	...	...	...	...	...	...	...	...	...	...	...	...
California	16.00	...	16.00	17.42	49.42	15	2	16.00	...	16.00	27.00	59.00	28	6

in the capital city of the State. The property taxes given for farm vehicles include State, county, and district or other property taxes that would be collected in an average rural agricultural community in the State. In a few States where uniform State-wide valuation and tax rates are in effect in all jurisdictions, there is no difference between property taxes on farm vehicles and those registered in the capital city of the State.

In two States—California and Washington—property taxes on motor vehicles have been replaced by "in lieu" taxes. These taxes have many characteristics of property taxes, but since neither the levies nor the distribution of their proceeds are directly related to the jurisdiction in which they were collected, they have been classified as road-user imposts. In both of these instances the State courts have ruled them not to be property taxes.

Types of Service

Distinctly different tax rates are in effect in most States for commercial vehicles operated privately (not for hire), as contract carriers (for hire), and in farm service. Information was therefore obtained, and was tabulated separately, for all vehicles in private operation, for the pickup and stake trucks (Nos. 3 and 4) in farm service, and for the stake and van trucks (Nos. 4 and 5) and all combinations (Nos. 7–11) in contract carrier service.

Vehicles in private operation are those used solely for carrying goods owned or sold by the vehicle owner, with no direct transportation charge.

Contract carriers are those employed in hauling goods for others, with a direct transportation charge, at times and to destinations required by the jobs for which they are hired. Operation of contract carriers is usually subject to State franchise and regulation. Common carriers, which operate for hire over established routes and on fixed schedules, are not included in the study because of their more complex tax schedules and forms of regulation, and the fact that their operation is predominantly interstate.

The farm service classification represents a reduced registration fee for farmers' trucks, but without restriction as to the highways they may use. This provision, in various forms, is in effect in 22 States. (The nominal charge imposed as the sole registration fee by some States for farm vehicles licensed for restricted operation on or in the immediate vicinity of the farm is not included in this study.)

Farm Service

Provision for registration of farm trucks at half the regular fee for private trucks is common but, as shown in tables 5 and 6, some States do not allow reductions that great, and some allow considerably more. The reductions for farm vehicles vary not only among the States but also vary somewhat for vehicles of different capacities, as may be seen from a comparison of figures 7 and 8 (p. 131). Large numbers of farm trucks are in the lighter groups, and what may at first glance appear to be relatively minor concessions in registration fees can, in fact, be of major importance from a

Table 6.—Road-user and personal-property taxes on a stake truck, 12,500 pounds gross vehicle weight (No. 4)

State	Farm service							Private operation							Contract carrier							
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Rank of State	
						Total fees and taxes	Total, excl. prop. tax						Total fees and taxes	Total, excl. prop. tax							Total fees and taxes	Total, excl. prop. tax
New England:																						
Maine	$60.00	$22.30		$33.00	$115.30	3	3	$60.00	$22.30		$79.98	$162.28	12	11	$60.00	$22.30		$5.00	$133.32	$220.62	20	23
New Hampshire	25.00	16.06		27.50	68.56	29	31	75.00	16.06		66.65	157.71	14	10	75.00	16.06		2.00	111.10	204.16	31	26
Vermont	32.00			27.50	59.50	38	24	118.75			66.65	185.40	4	2	118.75				111.10	229.85	19	15
Massachusetts	12.00	35.68		27.50	75.18	21	46	39.00	35.68		66.65	141.33	20	31	39.00	35.68		5.00	111.10	190.78	33	39
Rhode Island	39.00	28.00		22.00	89.00	11	19	39.00	41.16		53.32	133.48	25	41	39.00	41.16		5.00	88.88	174.04	38	46
Connecticut	37.50	44.33		22.00	103.83	4	22	37.50	44.33		53.32	135.15	22	43	37.50	44.33		5.00	88.88	175.71	37	47
Middle Atlantic:																						
New York	62.50			22.00	84.50	14	6	62.50			53.32	115.82	42	26	62.50				88.88	151.38	47	40
New Jersey	30.00		$1.00	16.50	47.50	47	41	60.00		$1.00	39.99	100.99	48	37	60.00		$1.00		66.66	127.66	48	48
Pennsylvania	45.00			27.50	72.50	25	11	45.00			66.65	111.65	45	29	45.00			64.00	111.10	220.10	21	17
Delaware	22.50			27.50	50.00	44	37	45.00			66.65	111.65	44	28	45.00				111.10	156.10	46	38
Maryland	10.00	7.00		27.50	44.50	48	47	35.00	15.00		66.65	116.65	40	35	35.00	15.00			111.10	161.10	44	41
District of Columbia	35.00	25.50		27.50	88.00	12	18	35.00	25.50		66.65	127.15	28	36	35.00	25.50			111.10	171.60	40	42
West Virginia	38.00	35.68		27.50	101.18	6	15	38.00	44.60		66.65	149.25	18	32	38.00	44.60		14.50	111.10	208.20	30	37
Southeastern:																						
Virginia	19.50	10.80		33.00	63.30	34	33	19.50	25.30		79.98	124.78	30	38	19.50	25.30		160.00	133.32	338.12	5	5
North Carolina	31.25	12.26		38.50	82.01	16	13	62.50	27.32		93.31	183.13	5	6	125.00	27.32			155.54	307.86	6	8
South Carolina	66.00	14.00		38.50	118.50	2	2	66.00	38.00		93.31	197.31	3	5	66.00	38.00		100.00	155.54	359.54	4	4
Georgia	10.00	14.53		33.00	57.53	40	44	10.00	32.03		79.98	122.01	34	45	25.00	32.03		25.00	133.32	215.35	23	29
Florida	53.00		.25	38.50	91.75	9	5	53.00		.25	93.31	146.56	19	9	145.25		.25	100.50	155.54	401.54	1	1
Kentucky	4.50	20.00	.50	38.50	63.50	33	43	32.00	40.30	.50	93.31	166.11	8	18	32.00	40.30	.50	77.00	155.54	305.34	7	9
Tennessee	12.50		.50	38.50	51.50	42	35	25.00		.50	93.31	118.81	39	24	50.00		.50	12.50	155.54	218.54	22	18
Alabama	22.50	14.70	.50	33.00	70.70	28	29	22.50	23.45	.50	79.98	126.43	29	33	22.50	23.45	.50	101.00	133.32	280.77	10	10
Mississippi	21.40	11.10		38.50	71.00	27	21	37.00	34.35		93.31	164.66	10	15	73.00	34.35		7.00	155.54	269.89	12	13
East Central:																						
Ohio	33.70			22.00	55.70	41	30	81.60			53.32	134.92	23	13	81.60			20.00	88.88	190.48	34	25
Indiana	35.00	45.00	.25	22.00	102.25	5	28	35.00	77.50	.25	53.32	166.07	9	46	35.00	77.50	.25	12.00	88.88	213.63	25	44
Illinois	86.00	14.07		27.50	127.57	1	1	86.00	15.08		66.65	167.73	7	8	86.00	15.08			111.10	212.18	26	24
Michigan	26.50			24.75	51.25	43	36	53.00			59.99	112.99	43	27	53.00			30.00	99.99	182.99	35	30
Wisconsin	27.50			22.00	49.50	45	38	110.00			53.32	163.32	11	4	110.00			106.00	88.88	304.88	8	6
Minnesota	32.40			27.50	59.90	36	20	50.00			66.65	116.65	41	25	50.00			7.50	111.10	168.60	42	35
Iowa	70.00			22.00	92.00	8	4	70.00			53.32	123.32	33	20	70.00			5.00	88.88	163.88	43	36
Missouri	50.00	8.29		16.50	74.79	22	14	50.00	17.04		39.99	107.03	46	44	50.00	17.04		25.00	66.66	158.70	45	43
Southwestern:																						
Arkansas	36.00	15.73		35.75	87.48	13	12	42.00	21.83		86.65	150.48	17	16	42.00	21.83			144.43	208.26	28	28
Louisiana	10.00			38.50	48.50	46	40	60.00			93.31	153.31	15	7	120.00			10.00	155.54	285.54	9	7
Oklahoma	22.40		.50	35.75	58.65	39	25	95.00		.50	86.65	182.15	6	3	95.00		.50		144.43	239.93	17	12
Texas	40.62	5.17		22.00	67.79	30	17	81.25	16.21		53.32	150.78	16	14	81.25	16.21		11.00	88.88	197.34	32	31
West Central:																						
North Dakota	32.00			27.50	59.50	37	23	32.00		21.00	66.65	119.65	37	22	32.00		21.00	50.00	111.10	214.10	24	19
South Dakota	37.50			27.50	65.00	31	16	37.50		95.00	66.65	199.15	1	1	37.50		95.00	10.00	111.10	253.60	14	11
Nebraska	4.00	31.42		27.50	62.92	35	48	60.00	70.96		66.65	197.61	2	17	60.00	70.96		10.00	111.10	252.06	16	32
Kansas	30.00	19.00		27.50	76.50	17	27	30.00	30.00	6.30	66.65	132.95	26	34	30.00	30.00		63.00	111.10	234.10	18	22
Mountain:																						
Montana	13.60	29.00		33.00	75.60	19	42	28.00	49.87		79.98	157.85	13	30	28.00	49.87		50.00	133.32	261.19	13	21
Idaho	40.00			33.00	73.00	23	10	40.00			79.98	119.98	35	21	40.00				133.32	173.32	39	34
Wyoming	15.00	21.41		27.50	63.91	32	45	15.00	21.41		66.65	103.06	47	48	15.00	21.41		60.70	111.10	208.21	29	27
Colorado	17.50	21.41	1.00	33.00	72.91	24	34	17.50	21.41	1.00	79.98	119.89	36	39	17.50	21.41	1.00	80.00	133.32	253.23	15	14
New Mexico	43.50			33.00	76.50	18	8	43.50			79.98	123.48	32	19	21.75			25.00	133.32	180.07	36	33
Arizona	30.00	32.20		27.50	89.70	10	26	30.00	32.20		66.65	128.85	27	40	30.00	32.20		200.00	111.10	373.30	3	3
Utah	25.00	19.00		27.50	71.50	26	32	25.00	32.86		66.65	124.51	31	42	25.00	32.86			111.10	168.96	41	45
Nevada	23.85	51.05		24.75	99.65	7	39	23.85	51.05		59.99	134.89	24	47	23.85	51.05		102.15	99.99	277.04	11	16
Pacific:																						
Washington	17.50		22.00	35.75	75.25	20	9	30.00		22.00	86.65	138.65	21	12	30.00		22.00	15.00	144.43	211.43	27	20
Oregon																						
California	36.00		23.00	24.75	83.75	15	7	36.00		23.00	59.99	118.99	38	23	36.00		23.00	230.00	99.99	388.99	2	2

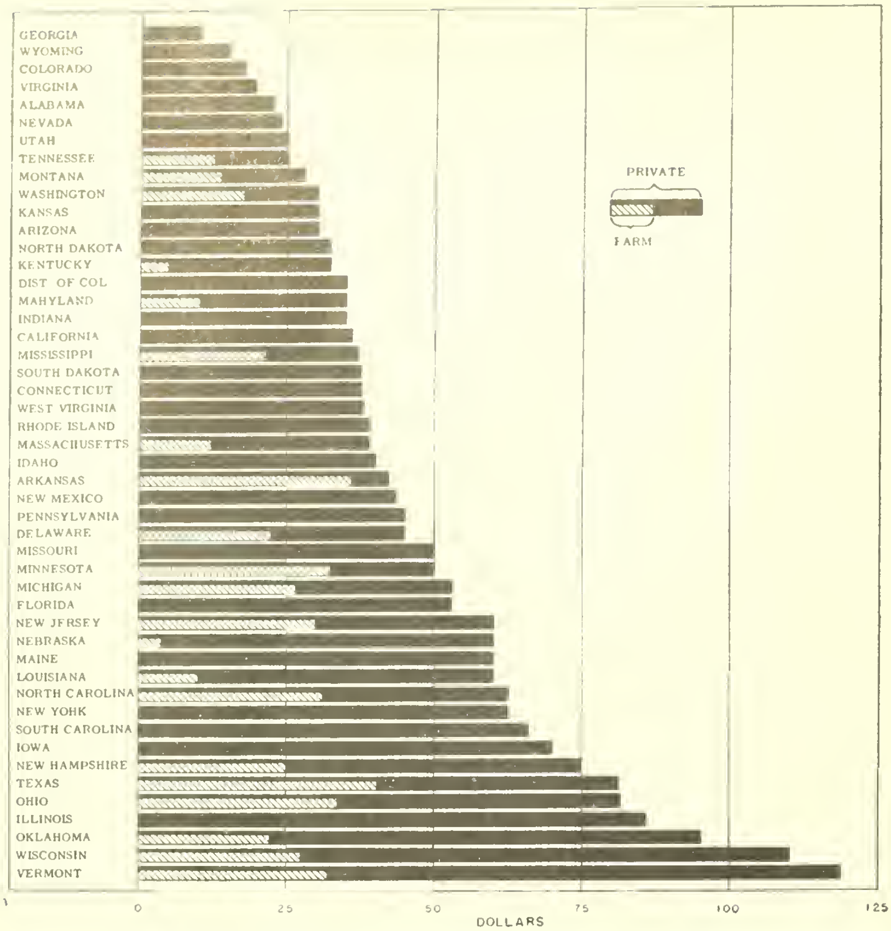


Figure 9.—Comparison of private operation and farm service registration fees on a 12,500-pound stake truck (No. 4).

revenue standpoint, particularly in predominantly agricultural States. In 4 States, as indicated in figure 8, the farm classification allows the 12,500-pound stake truck (No. 4) a reduction of more than 75 percent in registration fee, in 13 States the reduction is from 50 to 75 percent, and in 4 States, it is less than 50 percent. A comparison of the regular and farm registration fees for this vehicle in each State is shown in figure 9.

In Nebraska, the regular registration fee for the 12,500-pound stake truck is $60.00, but a farmer can register the same truck for $4.00. In Kentucky, a flat fee of $4.50 is applicable to farm trucks of 22,000 pounds or less, but a 22,000-pound vehicle in private use requires a registration fee of $134.00. The importance of these figures is underscored by the fact that in 1951 more than 54 percent of Nebraska's trucks, and in Kentucky, over 39 percent, were registered at reduced farm rates. Similar situations exist in other States, but in most the reductions are not as great. In New Hampshire, for example, a farm truck of less than 16,000 pounds may be registered at a flat fee of $25.00, but since this is greater than the regular fee of $21.15 for a pickup truck having a gross weight of 4,700 pounds, the farm rate is of no advantage to owners of the pickup. Consequently, only 5.2 percent of the total truck registrations in New Hampshire are in the farm classification.

In 1951, the farm truck registrations were more than half of all truck registrations in 5 States, and exceeded 30 percent in 13 States. It should not be forgotten, however, that the special reduced rates for registration of farm trucks are in part compensated for by the fact that farm trucks probably average fewer miles per year than other trucks. Thus the reductions are not as great on a cents-per-mile basis as it might appear from the comparisons in tables 5 and 6 and in figures 7 and 8.

Carrier Taxes

In the consideration of motor-carrier service, the gross receipts taxes and other levies that are imposed on general business, and not limited to motor carriers, have been eliminated. Fees for authority to operate, and other fees paid only at the time a carrier begins to haul goods for hire, have also been excluded.

Care should be taken in comparing the carrier taxes of the several States. Many of the States have special registration classes for vehicles used in contract (for-hire) service. The fees in the special registration classes, in most cases, are substantially greater than the registration fees for vehicles in private (not-for-hire) service. The difference, while technically classed as part of the registration fee, is in reality a special carrier fee. There is, of course, the compensating factor that vehicles in carrier service usually can be expected to operate a considerably greater mileage than those in private use, thus reducing the per-mile cost of their registration fees.

The majority of trucks are used for the ordinary private service of carrying goods owned or sold by the vehicle owner, and for which there is no direct transportation charge. Even these trucks, however, are subject to mileage or other carrier taxes under some conditions, and the variations in the tax structures of the States made it necessary in a few instances to make arbitrary decisions on the inclusion or exclusion of these special levies.

The general policy followed was to include only the taxes that have to be paid on almost all of the vehicles in a given group. For example, the laws of some States are so written that practically all vehicles of more than 1½ tons (manufacturer's rated capacity) have to pay certain mileage or compensatory taxes, whether in private or contract operation. In the same States many vehicles of 1½-ton capacity or less are not subject to the tax. In such a case the tax has been shown only for the heavier vehicles. It should be remembered that the purpose of the study is to present a comparison of tax rates and burdens on certain typical vehicles rather than to include all taxes on all vehicles. Undoubtedly many of the larger trucks traveling in interstate commerce are subject to greater taxes than are reported in this study, and are also subject to additional restrictions and regulations. The larger units apparently have certain offsetting economic advantages, however, or they would not be in use.

Mileage Taxes

The classification and treatment of mileage taxes is something of a problem. Historically, road-user taxes have been classed by most authorities into three major groups. The first and most important of these consists of fuel taxes and miscellaneous fees incidental to fuel taxation. The second major group of taxes, usually termed motor-vehicle revenues, consists of motor-vehicle registration fees and a group of additional fees, some of which are not paid annually, that include title fees, drivers' licenses, and other revenues of relatively minor importance. The third group of road-user revenues, motor-carrier taxes, consists of special taxes on for-hire carriers.

These three groups have in the past afforded adequate classification of road-user revenues, although there are borderline instances in which there was considerable question as to proper classification. Most common among these, of course, is the annual registration fee in States that levy higher plate fees on for-hire carriers. Should

Table 7.—Road-user and personal-property taxes on a single-unit van truck, 19,000 pounds gross vehicle weight (No. 5)

State	Private operation							Contract carrier							
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Rank of State	
						Total fees and taxes	Total, excl. prop. tax							Total fees and taxes	Total, excl. prop. tax
New England:															
Maine	$125.00	$60.74	....	$138.42	$324.16	7	14	$125.00	$60.74	....	$5.00	$230.76	$421.50	21	23
New Hampshire	114.00	43.73	....	115.35	273.08	21	21	114.00	43.73	....	2.00	192.30	352.03	33	33
Vermont	199.50	....	....	115.35	314.85	9	2	199.50	....	....	....	192.30	391.80	26	19
Massachusetts	57.00	97.18	....	115.35	269.53	24	41	57.00	97.18	....	5.00	192.30	351.48	34	43
Rhode Island	54.00	114.00	....	92.28	260.28	29	47	54.00	114.00	....	5.00	153.84	326.84	41	47
Connecticut	57.00	111.17	....	92.28	260.45	28	45	57.00	111.17	....	5.00	153.84	327.01	40	46
Middle Atlantic:															
New York	95.00	....	$90.00	92.28	277.28	18	9	95.00	....	....	150.00	153.84	398.84	23	15
New Jersey	90.00	....	1.00	69.21	160.21	48	44	90.00	....	$1.00	....	115.38	206.38	48	48
Pennsylvania	96.00	....	....	115.35	211.35	42	28	96.00	....	....	92.00	192.30	380.30	28	21
Delaware	71.00	....	....	115.35	186.35	46	36	71.00	....	....	....	192.30	263.30	46	40
Maryland	100.00	35.00	....	115.35	250.35	30	26	100.00	35.00	....	....	192.30	327.30	39	37
District of Columbia	65.00	56.50	....	115.35	236.85	35	40	65.00	56.50	....	....	192.30	313.80	43	42
West Virginia	84.50	121.48	....	115.35	321.33	8	29	84.50	121.48	....	23.50	192.30	421.78	20	35
Southeastern:															
Virginia	45.60	66.00	....	138.42	250.02	31	38	45.60	66.00	....	230.00	230.76	572.36	6	7
North Carolina	152.00	55.00	....	161.49	368.49	5	3	266.00	55.00	....	....	269.22	590.22	4	4
South Carolina	126.00	88.00	....	161.49	375.49	3	6	126.00	88 00	....	100.00	269.22	583.22	5	8
Georgia	50.00	74.58	....	138.42	263.00	26	34	100.00	74.58	....	25.00	230.76	430.34	18	25
Florida	86.00	....	.25	161.49	247.74	33	17	246.75	....	.25	250.50	269.22	766.72	1	1
Kentucky	112.00	97.93	.75	161.49	372.17	4	12	45.00	97.93	.50	121.00	269.22	533.65	7	11
Tennessee	80.00	....	.50	161.49	241.99	34	18	140.00	....	.50	12.50	269.22	422.22	19	12
Alabama	50.00	43.50	.50	138.42	232.42	37	33	50.00	43.50	.50	126.00	230.76	450.76	15	14
Mississippi	94.00	54.19	....	161.49	309.68	10	16	187.00	54.19	....	13.00	269.22	523.41	9	10
East Central:															
Ohio	168.80	....	....	92.28	261.08	27	15	168.80	....	....	20.00	153.84	342.64	37	30
Indiana	50.00	156.16	.25	92.28	298.69	12	48	50.00	156.16	.25	12.00	153.84	372.25	29	45
Illinois	159.00	18.25	....	115.35	292.60	14	11	159.00	18.25	....	....	192.30	369.55	30	27
Michigan	129.00	....	....	103.82	232.82	36	20	129.00	....	....	50 00	173.07	352.07	32	26
Wisconsin	185.00	....	....	92.28	277.28	19	10	185.00	....	....	190.00	153.84	528.84	8	5
Minnesota	80.00	....	....	115.35	195.35	44	31	80.00	....	....	7.50	192.30	279.80	45	39
Iowa	190.00	....	....	92.28	282.28	16	7	190.00	....	....	5.00	153.84	348.84	36	28
Missouri	80.00	30.08	....	69.21	179.29	47	46	80.00	30.08	....	25.00	115.38	250.46	47	44
Southwestern:															
Arkansas	67.00	32.10	....	149.96	249.06	32	24	67.00	32.10	....	....	249.99	349.09	35	31
Louisiana	120.00	....	....	161.49	281.49	17	8	240.00	....	....	10.00	269.22	519.22	10	6
Oklahoma	145.00	....	.50	149.96	295.46	13	5	145.00	....	.50	....	249.99	395.49	24	17
Texas	133.00	42.22	....	92.28	267.50	25	23	133.00	42.22	....	11.00	153.84	340.06	38	36
West Central:															
North Dakota	44.00	....	30.00	115.35	189.35	45	32	44.00	....	30.00	50.00	192.30	316.30	42	32
South Dakota	100.00	....	185.00	115.35	400.35	2	1	100.00	....	185.00	10.00	192.30	487.30	12	9
Nebraska	100.00	134.56	....	115.35	349.91	6	27	100.00	134.56	....	10.00	192.30	436.86	17	34
Kansas	100.00	60.00	23.75	115.35	299.10	11	19	100.00	60.00	....	118.75	192.30	471.05	13	13
Mountain:															
Montana	50.00	99.11	....	138.42	287.53	15	35	50.00	99.11	....	67.50	230.76	447.37	16	29
Idaho	60.00	....	....	138.42	198.42	43	30	60.00	....	....	....	230.76	290.76	44	38
Wyoming	50.00	58.31	....	115.35	223.66	39	42	50.00	58.31	....	115.31	192.30	415.92	22	24
Colorado	25.00	58.31	1.00	138.42	222.73	40	43	25.00	58.31	1.00	142.00	230.76	457.07	14	16
New Mexico	78.00	....	....	138.42	216.42	41	25	39.00	....	....	125.00	230.76	394.76	25	18
Arizona	68.00	87.40	....	115.35	270.75	22	39	68.00	87.40	....	287.50	192.30	635.20	2	3
Utah	70.00	91.14	....	115.35	276.49	20	37	70.00	91.14	....	....	192.30	353.44	31	41
Nevada	38.70	139.06	153.30	103.82	434.88	1	4	38.70	139.06	....	153.30	173.07	504.13	11	22
Pacific:															
Washington	75.00		45.25	149.96	270.21	23	13	75.00	....	45.25	18.00	249.99	388.24	27	20
Oregon	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
California	66.00	....	56.00	103.82	225.82	38	22	66.00	....	56.00	325.00	173.07	620.07	3	2

the difference between the registration fee in private operation and for-hire operation be classed as a carrier tax? It has not been in the past, under the theory that carriers' vehicles as a rule can be expected to operate more miles than vehicles in private operation, and that the higher registration fee for carriers' vehicles serves as an adjustment for the greater mileage.

Another instance of borderline classification is the special fee charged to diesel-powered vehicles in the States that do not tax diesel fuel. An argument can be made for classification of these additional registration fees as fuel taxes. But the important point is the fairness of the State's tax system, and this is judged primarily on the total taxes levied on any vehicle, provided, of course, that the methods of reporting or collecting the tax do not in themselves impose additional burdens of record keeping or other requirements not reflected in the actual amounts of taxes paid.

We may be witnessing the development of a field of highway taxation—mileage and ton-mile taxes—that should not be placed in any of the three historical groups. It does not appear that the mileage taxes in Idaho, New York, and Oregon can be classed as fuel taxes, as registration fees, or as carrier taxes. They are avowed attempts to obtain specific amounts of revenues based almost entirely on the vehicles and their use and make no distinction between operation for profit as a transportation facility, and operation incidental to the conduct of any other enterprise. It is not suggested at present that a separate classification of mileage taxes be established, but it is time to put forth the question for discussion. Several States have shown great interest in mileage taxes and, if the movement spreads, those interested in highway finance will be required to give it considerable attention.

Mileage taxes may prove to have two great advantages: They should yield considerable amounts of revenue, and they should, in theory at least, be imposed at rates that measure value of service received, to be applied toward the costs of providing the service. Undoubtedly the mileage tax will also prove to have at least two major offsetting disadvantages. The most important of these from the States' point of view is the cost of administration: The cost of establishing and maintaining complete records, weight stations, and enforcement staff will make any State considering a mileage tax look twice. For the vehicle owner, the maintenance of records for filing of reports necessary under a mileage tax will certainly constitute a substantial burden over and above the actual amount of tax paid.

Administration and Application

While this study is based primarily on legislation, it is necessary to call attention to administration and enforcement as they affect vehicle taxation. The interpretation and application of road-user taxes in one State as compared to another with similar tax laws might easily result in substantial tax differences on similar vehicles in similar service. Rigid insistence that vehicles be registered for the full weights they carry (not to be confused with size and weight

Table 8.—Road-user and personal-property taxes on a single-unit, three-axle dump truck, 40,000 pounds gross vehicle weight (No. 6)

State	Private operation						
						Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:							
Maine	$300.00	$140.88		$450.00	$890.88	11	15
New Hampshire	240.00	101.43		375.00	716.43	30	31
Vermont	420.00	...		375.00	795.00	18	11
Massachusetts	120.00	225.40		375.00	720.40	29	44
Rhode Island	127.00	265.00		300.00	692.00	34	46
Connecticut	200.00	257.86		300.00	757.86	25	42
Middle Atlantic:							
New York	200.00	...	$187.50	300.00	687.50	35	23
New Jersey	240.00		1.00	225.00	466.00	46	45
Pennsylvania	250.00	...		375.00	625.00	41	28
Delaware	155.00	...		375.00	530.00	45	35
Maryland	200.00	50.00		375.00	625.00	40	33
District of Columbia	150.00	132.00		375.00	657.00	39	39
West Virginia	242.00	281.75		375.00	898.75	10	30
Southeastern:							
Virginia	180.00	150.48		450.00	780.48	20	27
North Carolina	320.00	165.67		525.00	1,010.67	6	7
South Carolina	351.00	209.00		525.00	1,085.00	4	4
Georgia	50.00	206.24	...	450.00	706.24	33	43
Florida	189.00	...	.25	525.00	714.25	32	21
Kentucky	350.00	227.09	.75	525.00	1,102.84	3	5
Tennessee	275.00	...	.50	525.00	800.50	17	10
Alabama	50.00	101.84	.50	450.00	602.34	43	40
Mississippi	...	...	...	...	...	...	...
East Central:							
Ohio	492.25	...	...	300.00	792.25	19	12
Indiana	200.00	471.95	.25	300.00	972.20	8	41
Illinois	640.00	56.25	...	375.00	1,071.25	5	2
Michigan	378.00	...		337.50	715.50	31	20
Wisconsin	460.00	...		300.00	760.00	24	14
Minnesota	350.00	...		375.00	725.00	28	19
Iowa	465.00	...		300.00	765.00	22	13
Missouri	300.00	48.12		225.00	573.12	44	37
Southwestern:							
Arkansas	200.00	40.18		487.50	727.68	26	22
Louisiana	200.00	...	...	525.00	725.00	27	18
Oklahoma	395.00	...	.50	487.50	883.00	12	3
Texas	360.00	103.17	...	300.00	763.17	23	26
West Central:							
North Dakota	400.00	...	100.00	375.00	875.00	13	6
South Dakota	412.50	...	465.00	375.00	1,252.50	1	1
Nebraska	220.00	353.46	...	375.00	948.46	9	32
Kansas	250.00	140.00	100.00	375.00	865.00	14	17
Mountain:							
Montana	300.00	230.84	...	450.00	980.84	7	16
Idaho	...	...	394.20	450.00	844.20	16	8
Wyoming	150.00	135.24	...	375.00	660.24	38	38
Colorado	75.00	135.24	1.00	450.00	661.24	37	36
New Mexico	172.50	...	...	450.00	622.50	42	29
Arizona	188.50	202.80		375.00	766.30	21	34
Utah	295.00	177.32	...	375.00	847.32	15	24
Nevada	85.05	322.55	398.00	337.50	1,143.10	2	9
Pacific:							
Washington	...	...	...	...	...	...	...
Oregon	...	...	...	...	...	...	...
California	206.00	...	126.00	337.50	669.50	36	25

limit enforcement), would undoubtedly increase the registration revenues of some States. Practices are understood to vary considerably in this respect. It is necessary, therefore, that any careful comparison of road-user taxes be accompanied by a close examination of practices and interpretations in the application of existing statutes. The data presented in this study make no allowances for such differences.

The payment of motor-fuel taxes is another item that merits close examination. Practically all persons familiar with highway finance agree that some of the gasoline used on the highways escapes taxation (by means of refunded taxes), and that the amounts vary greatly between States. There is no way of knowing how much revenue escapes through excessive refund claims, but it is probable that the total is considerable.

For example, the stipulation in this study for the light-weight automobile is 9,500 miles of annual travel at 16.5 miles per gallon, with total gasoline consumption of 576 gallons for the year. Yet the average for *all* vehicles in one State was less than 500 taxed gallons in both 1950 and 1951. Traffic counts and other information indicate that the residents operate their vehicles about as much as residents of other States. In view of the amazing "fuel economy" of their vehicles, it appears that taxation of ownership, as distinguished from taxation of use, is a surer source of revenue under such conditions. (The State legislature apparently made this deduction some time ago, and imposed registration fees of $25.00 and $35.00 on light and medium-weight passenger cars, respectively, and eliminated reductions for farm trucks.)

Another subject for scrutiny is the collection of diesel-fuel taxes. There are numerous ways of avoiding or evading the tax on diesel fuel, and the State authorities charged with collecting it are in practically unanimous agreement that a considerable amount escapes taxation or is uncollectible. These authorities feel, however, that the situation is improving.

Summary of Comparisons

The foregoing discussion concerns general subjects rather than specific vehicles, except for the remarks on farm rates. It is impractical to include in this report a full discussion of each of the tables and charts presented, but there are comparisons and items of special interest that do merit being called to attention in connection with each of the vehicles.

Vehicle No. 1, light-weight passenger car (table 4; fig. 12).—Registration fees for the light-weight passenger car range from $3.00 to $28.76, total road-user taxes from $28.28 to $66.70. Since this vehicle is assumed to use 576 gallons of gasoline a year, each cent of a State's gasoline tax rate amounts to a levy of $5.76 during the course of a year, or 11 cents a week. The lowest gasoline tax rate costs the owner of the light passenger car 33 cents a week, the highest about 78 cents a week.

There are 15 States with property taxes alone that exceed the total road-user taxes paid in the lowest State. (The property taxes shown on these vehicles are for the State capitals, however, and undoubtedly exceed the averages for the States in most cases.)

There are 25 States in which the total road-user taxes on this light automobile amount to less than $40 a year, or 80 cents a week. In 11 of the States, road-user taxes amount to between $40 and $50, and in 10 States they are between $50 and $60. In only two States do they exceed $60.

Vehicle No. 2, medium-weight passenger car (table 4).—The medium- and light-weight passenger cars are representative of far more than half of all vehicles on the highways. Although the taxation of automobiles probably receives less attention than the taxation of heavier commercial units, automobiles, because of their vast numbers, contribute substantially more in road-user taxes.

There are not great differences between the road-user taxes on light automobiles and those of medium weight. The registration fees for this vehicle range from $3.00 to $37.26. The $37.26 fee applies in Oklahoma and this registration fee combined with a 6.5-cent gasoline tax makes the total road-user taxes $84.82, the highest for this vehicle in any State. However, there is no property tax on motor vehicles in Oklahoma and if property taxes are included for States in which they are imposed, Oklahoma ranks 18th in total fees and taxes. Nebraska, on the other hand, with a $5.00 registration fee and a 5-cent gasoline tax, obtains a total of $41.20 in road-user taxes on the medium-weight sedan, and ranks 40th among the States. If the property tax of $74.51 is added, however, the total becomes $115.71, the highest in any State.

Vehicle No. 3, pickup truck (table 5; fig. 13).—The registration fees for the pickup truck in private use are slightly higher than those on passenger cars in most States. However, large numbers of pickup trucks are registered at reduced farm rates. The lowest registration fee on any vehicle covered in the study, $2.50, applies to pickup trucks in Georgia regardless of whether they are in farm or private service. The highest registration fee on the pickup truck, $34.80, was found in Ohio. It is probable that pickup trucks outnumber other trucks in many States. Consequently, although the total taxes on each pickup are relatively small compared to taxes on heavier vehicles, the large number of pickups makes this an extremely important category from a revenue standpoint.

Vehicle No. 4, stake truck, 12,500 pounds gross vehicle weight (table 6; fig. 14).—Vehicles in the 12,500-pound group are subject to registration fees that in some States break away rather sharply from the fees charged on automobiles and pickup trucks, although this is not true in all cases. The stake truck may be registered in Wyoming for $15.00, in Colorado for $17.50, and in Virginia for $19.50. In Vermont and Wisconsin, however, the fees are $118.75 and $110.00, respectively, for registration in private service. The farm rates, as discussed elsewhere, offer vehicles substantial reductions in many States, including Vermont and Wisconsin.

In contrast to the heavier trucks, there are large numbers of vehicles registered in the approximate weight range represented by this 12,500-pound unit (the group commonly classed as "1½-ton").

Vehicle No. 5, van truck, 19,000 pounds gross vehicle weight (table 7; fig. 15).—The lowest registration fee on the 19,000-pound van truck, $25.00, is found in Colorado, and the highest, $199.50, is found in Vermont. Although this van can be registered in South Dakota for $100.00 (slightly below the average of $104.65 for all States) the inclusion of the mileage tax of $185.00 in South Dakota makes a total of $400.35 in road-user taxes, the highest found in any State on this vehicle.

Vehicle No. 6, 40,000-pound dump truck (table 8; fig. 16).—The 40,000-pound dump truck, with perhaps minor adjustments of axle spacing, would be permitted to operate in all States except Mississippi and Washington. (Its axle spacing and loading are representative of concrete transit-mix and other special equipment.) The lowest registration fees on this vehicle are $50.00 in Alabama and Georgia and $75.00 in Colorado. These are the only States that still register trucks on the basis of manufacturer's rated capacity. The highest fee, $640.00, is found in Illinois, and the average for all States is $295.20. Although the Illinois registration fee is higher than that of any other State on this vehicle, the total road-user taxes on it would be greater in South Dakota, where the mileage tax applies.

Vehicle No. 7, 40,000-pound combination (table 10; fig. 17).—The 40,000-pound combination is typical of the largest group of combinations, and is found in substantial numbers even where much heavier vehicles are permitted. It is legal in all States. The average road-user tax payment on it in private operation is $677.50. The lowest registration fee on this combination, $45.00, is found in Colorado; the highest, $640.00, in Illinois. The average registration fee is $255.83, somewhat lower than on the three-axle single-unit truck (No. 6) of the same gross weight.

As an interesting sidelight, table 9 shows for this combination, in private operation, the taxes levied on the tractor-truck and on the semitrailer as individual units. The ratio of the taxes on the component units varies widely among the States: in Minnesota, for example, the $10.00 tax on the semitrailer is little more than 1 percent of the $750.00 levied on the tractor, while in Oklahoma the $295.50 for the semitrailer is 50 percent of the $585.50 for the power unit.

Table 9.—Road-user and personal-property taxes on the tractor and the semitrailer of a three-axle combination, 40,000 pounds gross vehicle weight (No. 7), in private operation

State	Tractor-truck					Semitrailer			
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Registration fee	Property tax	Other taxes and fees	Total
New England:									
Maine	$300.00	$76.28		$480.00	$856.28	$5.00	$75.00		$80.00
New Hampshire	240.00	94.52		400.00	734.52				
Vermont	420.00			400.00	820.00	15.00			15.00
Massachusetts	120.00	122.04		400.00	642.04	2.00	88.00		90.00
Rhode Island	127.00	144.00		320.00	591.00	2.00	103.00		105.00
Connecticut	200.00	240.29		320.00	760.29				
Middle Atlantic:									
New York	88.25		$370.00	320.00	778.25	90.00			90.00
New Jersey	110.00		1.00	240.00	351.00	90.00		$1.00	91.00
Pennsylvania	120.00			400.00	520.00	75.00			75.00
Delaware	83.00			400.00	483.00	67.00			67.00
Maryland	35.00	30.00		400.00	465.00	100.00	15.00		115.00
District of Columbia	65.00	75.50		400.00	540.50	50.00	70.40		120.40
West Virginia	227.00	152.55		400.00	779.55	15.00	110.00		125.00
Southeastern:									
Virginia	30.00	82.28		480.00	592.28	150.00	59.40		209.40
North Carolina	160.00	72.28		560.00	792.28	160.00	68.60		228.60
South Carolina	66.00	100.00		560.00	726.00	96.00	77.00		173.00
Georgia	20.00	52.16		480.00	552.16	100.00	80.52		180.52
Florida	88.00		.25	560.00	648.25	109.50		.25	109.75
Kentucky	350.00	122.96	.75	560.00	1,033.71		88.66		88.66
Tennessee	275.00		.50	560.00	835.50				
Alabama	50.00	50.25	.50	480.00	580.75	25.00	40.20	.50	65.70
Mississippi	271.00	61.86		560.00	892.86	11.00	45.75		56.75
East Central:									
Ohio	177.20			320.00	497.20	135.20			135.20
Indiana	190.00	204.75	.25	320.00	715.00	25.00	166.57		191.57
Illinois	640.00	78.75		400.00	1,118.75				
Michigan	154.00			360.00	514.00	127.75			127.75
Wisconsin	60.00			320.00	380.00	167.50			167.50
Minnesota	350.00			400.00	750.00	10.00			10.00
Iowa	435.00			320.00	755.00	60.00			60.00
Missouri	300.00	32.08		240.00	572.08	7.00	16.04		23.04
Southwestern:									
Arkansas	200.00	34.56		520.00	754.56	5.00	29.40		34.40
Louisiana	140.00			560.00	700.00	120.00			120.00
Oklahoma	65.00		.50	520.00	585.50	295.00		.50	295.50
Texas	154.00	56.00		320.00	530.00	117.00	45.46		162.46
West Central:									
North Dakota	400.00		100.00	400.00	900.00				
South Dakota	162.50		215.00	400.00	777.50	60.00		185.00	245.00
Nebraska	240.00	160.76		400.00	800.76	1.00	111.36		112.36
Kansas	30.00	83.00	200.00	400.00	713.00	100.00	55.00		155.00
Mountain:									
Montana	60.00	124.82		480.00	664.82	32.50	90.03		122.53
Idaho	65.00			480.00	545.00	54.00			54.00
Wyoming	50.00	73.22		400.00	523.22	40.00	52.80		92.80
Colorado	25.00	73.22	1.00	480.00	579.22	20.00	52.80		72.80
New Mexico	88.50			480.00	568.50	74.00			74.00
Arizona	69.50	109.80		400.00	579.30	50.95	79.20		130.15
Utah	70.00	91.76		400.00	561.76	90.00	81.84		171.84
Nevada	39.60	174.64	156.40	360.00	730.64	32.85	125.93	133.15	291.93
Pacific:									
Washington	105.00		46.25	520.00	671.25	55.00		39.50	94.50
Oregon	......	......	......	......	......	......	......	......	......
California	66.00		71.00	360.00	497.00	81.00		52.00	133.00

Vehicle No. 8, 50,000-pound gasoline-powered combination (table 11; fig. 18).—The 50,000-pound combination is legal in all States except Pennsylvania, Kentucky, and Tennessee, and can be operated in those States with reduced loads. The average registration fee for this combination is $341.17 when in private service, but in contract service the average registration fee is $586.70. The highest annual total of road-user taxes in private service is $1,637.50, but in contract service the highest total of road-user taxes is $2,555.25.

Figure 10 (p. 143) compares, for this combination and for the 40,000-pound combination, the tax costs in cents per mile of private and contract operation.

Vehicle No. 9, 50,000-pound diesel-powered combination (table 12; fig. 19).—The 50,000-pound diesel-powered combination, empty, is somewhat heavier than the 50,000-pound gasoline combination and thus carries a correspondingly smaller payload when at full capacity. (It is also a somewhat more expensive combination than the one with gasoline power and is subject to greater property taxes in many States.) Offset against this, however, is its substantial saving in total fuel cost, including tax. The diesel combination is assumed to save one-third in fuel gallonage. The saving in fuel is highly desirable from almost any standpoint, but the resulting reductions in

Table 10.—Road-user and personal-property taxes on a three-axle tractor-semitrailer combination, 40,000 pounds gross vehicle weight (No. 7)

State	Private operation							Contract carrier							
						Rank of State								Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excl. Property tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excl. prop tax
New England:															
Maine	$305.00	$151.28	......	$480.00	$936.28	7	13	$305.00	$151.28	......	$10.00	$480.00	$946.28	21	25
New Hampshire	240.00	94.52	......	400.00	734.52	27	26	240.00	94.52	......	2.00	400.00	736.52	38	36
Vermont	435.00	......	......	400.00	835.00	17	10	435.00	......	......	......	400.00	835.00	29	22
Massachusetts	122.00	210.04	......	400.00	732.04	30	42	122.00	210.04	......	5.00	400.00	737.04	37	44
Rhode Island	129.00	247.00	......	320.00	696.00	32	47	129.00	247.00	......	5.00	320.00	701.00	41	47
Connecticut	200.00	240.29	......	320.00	760.29	24	44	200.00	240.29	......	5.00	320.00	765.29	35	45
Middle Atlantic:															
New York	178.25	......	$370.00	320.00	868.25	14	7	178.25	......	......	370.00	320.00	868.25	28	19
New Jersey	200.00	......	2.00	240.00	442.00	48	48	200.00	......	$2.00	......	240.00	442.00	48	48
Pennsylvania	195.00	......	......	400.00	595.00	44	31	195.00	......	......	184.00	400.00	779.00	33	26
Delaware	150.00	......	......	400.00	550.00	46	36	150.00	......	......	......	400.00	550.00	47	42
Maryland	135.00	45.00	......	400.00	580.00	45	40	135.00	45.00	......	......	400.00	580.00	46	43
District of Columbia	115.00	145.90	......	400.00	660.90	34	45	115.00	145.90	......	......	400.00	660.90	43	46
West Virginia	242.00	262.55	......	400.00	904.55	10	23	242.00	262.55	......	38.00	400.00	942.55	22	33
Southeastern:															
Virginia	180.00	141.68	......	480.00	801.68	20	21	180.00	141.68	......	460.00	480.00	1,261.68	3	6
North Carolina	320.00	140.88	......	560.00	1,020.88	5	6	560.00	140.88	......	......	560.00	1,260.88	4	5
South Carolina	162.00	177.00	......	560.00	899.00	12	20	162.00	177.00	......	350.00	560.00	1,249.00	5	11
Georgia	120.00	132.68	......	480.00	732.68	29	29	237.50	132.68	......	25.00	480.00	875.18	27	29
Florida	197.50	......	.50	560.00	758.00	26	16	422.50	......	.50	200.50	560.00	1,183.50	9	2
Kentucky	350.00	211.62	.75	560.00	1,122.37	1	3	150.00	211.62	.50	320.00	560.00	1,242.12	6	14
Tennessee	275.00	......	.50	560.00	835.50	16	9	400.00	......	.50	17.50	560.00	978.00	18	15
Alabama	75.00	90.45	1.00	480.00	646.45	36	35	75.00	90.45	1.00	301.00	480.00	947.45	20	21
Mississippi	282.00	107.61	......	560.00	949.61	6	8	552.00	107.61	......	13.00	560.00	1,232.61	7	4
East Central:															
Ohio	312.40	......	......	320.00	632.40	39	27	312.40	......	......	30.00	320.00	662.40	42	34
Indiana	215.00	371.32	.25	320.00	906.57	9	39	215.00	371.32	.25	24.00	320.00	930.57	24	41
Illinois	640.00	78.75	......	400.00	1,118.75	2	1	640.00	78.75	......	......	400.00	1,118.75	11	12
Michigan	281.75	......	......	360.00	641.75	38	24	281.75	......	......	120.00	360.00	761.75	36	28
Wisconsin	227.50	......	......	320.00	547.50	47	37	227.50	......	......	560.00	320.00	1,107.50	12	7
Minnesota	360.00	......	......	400.00	760.00	25	15	360.00	......	......	7.50	400.00	767.50	34	27
Iowa	495.00	......	......	320.00	815.00	19	12	495.00	......	......	5.00	320.00	820.00	30	23
Missouri	307.00	48.12	......	240.00	595.12	43	38	307.00	48.12	......	25.00	240.00	620.12	44	39
Southwestern:															
Arkansas	205.00	63.96	......	520.00	788.96	21	18	205.00	63.96	......	......	520.00	788.96	32	30
Louisiana	260.00	......	......	560.00	820.00	18	11	520.00	......	......	10.00	560.00	1,090.00	13	10
Oklahoma	360.00	......	1.00	520.00	881.00	13	5	360.00	......	1.00	......	520.00	881.00	26	18
Texas	271.00	101.46	......	320.00	692.46	33	32	271.00	101.46	......	11.00	320.00	703.46	40	37
West Central:															
North Dakota	400.00	......	100.00	400.00	900.00	11	4	400.00	......	100.00	50.00	400.00	950.00	19	16
South Dakota	222.50	......	400.00	400.00	1,022.50	4	2	222.50	......	400.00	10.00	400.00	1,032.50	15	13
Nebraska	241.00	272.12	......	400.00	913.12	8	25	241.00	272.12	......	20.00	400.00	933.12	23	35
Kansas	130.00	138.00	200.00	400.00	868.00	15	17	130.00	138.00	......	400.00	400.00	1,068.00	14	17
Mountain:															
Montana	92.50	214.85	......	480.00	787.35	22	33	92.50	214.85	......	125.00	480.00	912.35	25	32
Idaho	119.00	......	......	480.00	599.00	42	30	119.00	......	......	......	480.00	599.00	45	38
Wyoming	90.00	126.02	......	400.00	616.02	41	46	90.00	126.02	......	369.50	400.00	985.52	17	20
Colorado	45.00	126.02	1.00	480.00	652.02	35	41	45.00	126.02	1.00	570.00	480.00	1,222.02	8	8
New Mexico	162.50	......	......	480.00	642.50	37	22	81.25	......	......	600.00	480.00	1,161.25	10	3
Arizona	120.45	189.00	......	400.00	709.45	31	43	120.45	189.00	......	575.00	400.00	1,284.45	1	9
Utah	160.00	173.60	......	400.00	733.60	28	34	160.00	173.60	......	......	400.00	733.60	39	40
Nevada	72.45	300.57	289.55	360.00	1,022.57	3	19	72.45	300.57	......	289.55	360.00	1,022.57	16	31
Pacific:															
Washington	160.00	......	85.75	520.00	765.75	23	14	160.00	......	85.75	36.00	520.00	801.75	31	24
Oregon	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
California	147.00	......	123.00	360.00	630.00	40	28	147.00	......	123.00	645.00	360.00	1,275.00	2	1

fuel tax contributions for the support of highways poses a problem. Michigan, Mississippi, New York, and Texas have already imposed higher rates on diesel fuel in an effort to obtain a tax contribution similar to that of gasoline-powered vehicles. Vermont and Nebraska do not tax diesel fuel but do impose equalization fees. Idaho does not levy any tax on diesel fuel consumed by vehicles subject to the mileage tax, but the mileage tax on diesel-powered vehicles is higher than on gasoline-powered vehicles. The relation between taxes paid on a gasoline-powered combination of 50,000 pounds gross weight and a similar diesel-powered combination may be developed by comparison of data in tables 11 and 12 (upper section).

Vehicle No. 10, 64,000-pound combination, and vehicle No. 11, 72,000-pound combination (table 12; figs. 20 and 21).—The heavy combinations are permitted in relatively few States. Where they are permitted, the taxes imposed are sometimes related to the effect of the vehicles on highway capacity, bridge load limits, acceptability of the vehicles from a traffic standpoint, and other factors not entirely concerned with the need for revenue. Since the taxation of such heavy combinations poses special problems in each State, further discussion is omitted here.

Comparisons between the taxes on private and contract carriers on a cents-per-mile basis are shown in figure 11 (p. 143).

Some Observations

Although the essential value of this study lies in the tables and charts, there are several items that merit special mention. These cannot be classed as new information, but they are put into perspective and measured.

Types of service.—Aside from the substantial differences in the computation of road-user taxes in the several States, there are very great differences in the levies on similar vehicles in different service. The vehicle of the private operator imposes the same burdens on the highway as the corresponding vehicle of the for-hire carrier, yet he does not pay nearly as much per mile in taxes in most States. It is notable that there is no difference between the taxation of for-hire and private carriers under the new mileage taxes in Idaho and New York.

Farm rates.—The registration fee reductions for farm trucks are substantial. It is doubtful whether these allowances can be fully justified from a highway finance standpoint, even though farm vehicles may travel fewer miles than other vehicles. The justification therefore must come under the heading of public policy and any State with farm rate reductions should carefully examine their relationship to other registration fees.

Diesel vehicles.—There is much evidence that, in general, diesel vehicles are making smaller tax contributions for highways in most States than corresponding gasoline-powered vehicles. The diesel vehicle is commonly assumed to use about two-thirds as much fuel as its gasoline-powered

Table 11.—Road-user and personal-property taxes on a gasoline-powered, four-axle, tractor-semitrailer combination, 50,000 pounds gross vehicle weight (No. 8)

State	Private operation							Contract carrier							
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Rank of State	
						Total fees and taxes	Total, excl. property tax							Total fees and taxes	Total, excl. prop. tax
New England:															
Maine	$355 00	$192.38	...	$975.00	$1,522.38	10	15	$355 00	$192 38	...	$10.00	$975 00	$1,532.38	22	24
New Hampshire	300.00	118.91	...	812.50	1,231.41	25	26	300 00	118.91	...	2 00	812 50	1,233.41	33	32
Vermont	540.00	...	...	812.50	1,352.50	20	14	540 00	...	...	...	812.50	1,352.50	28	23
Massachusetts	152.00	264.24	...	812.50	1,228.74	26	37	152.00	264.24	...	5 00	812 50	1,233.74	32	40
Rhode Island	152.00	310.00	...	650.00	1,112 00	35	44	152.00	310.00	...	5.00	650 00	1,117.00	39	44
Connecticut	250 00	302.29	...	650.00	1,202.29	27	43	250.00	302.29	...	5.00	650.00	1,207.29	35	43
MiddleAtlantic:															
New York	214.00	...	$747 50	650.00	1,611.50	6	2	214.00	...	...	747.50	650.00	1,611 50	18	14
New Jersey	260.00	...	2.00	487.50	749.50	45	45	260.00	...	$2.00	...	487 50	749.50	45	45
Pennsylvania	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Delaware	190.00	...	...	812.50	1,002 50	42	32	190.00	...	...	...	812 50	1,002 50	44	37
Maryland	210 00	55 00	...	812.50	1,077.50	37	31	210 00	55 00	...	...	812.50	1,077.50	42	34
Dist. of Columbia	130.00	185.40	...	812.50	1,127.90	33	39	130.00	185 40	...	...	812 50	1,127 90	37	41
West Virginia	317.00	330.30	...	812.50	1,459.80	14	23	317.00	330 30	...	47.00	812.50	1,506 80	23	29
Southeastern:															
Virginia	300.00	177.98	...	975.00	1,452.98	15	17	300.00	177.98	...	1,040.00	975 00	2,492.98	3	3
North Carolina	400.00	139.28	...	1,137.50	1,676.78	3	7	700.00	139 28	...	...	1,137.50	1,976.78	8	8
South Carolina	282.00	237.00	...	1,137.50	1,656 50	4	11	282.00	237.00	...	400.00	1,137.50	2,056 50	5	10
Georgia	170.00	202.40	...	975.00	1,347.40	21	20	337.50	202.40	...	25.00	975 00	1,539.90	21	25
Florida	214.50	...	.50	1,137.50	1,352 50	19	13	391.00	...	.50	325.50	1,137.50	1,854.50	10	6
Kentucky	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Tennessee	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Alabama	75.00	119.73	1.00	975 00	1,170.73	31	27	75.00	119 73	1.00	488.50	975.00	1,659.23	16	19
Mississippi	333.00	132.06	...	1,137.50	1,602 56	7	9	654 00	132 06	...	13.00	1,137.50	1,936.56	9	11
East Central:															
Ohio	334.80	...	...	650 00	984.80	43	36	334.80	...	...	30 00	650.00	1,014 80	43	36
Indiana	265.00	366.69	.25	650.00	1,281.94	23	41	265.00	366.69	25	24.00	650 00	1,305.94	30	42
Illinois	789.00	84.69	...	812.50	1,686.19	2	3	789 00	84 69	...	...	812 50	1,686.19	14	15
Michigan	299.25	...	...	731.25	1,030.50	41	30	299.25	...	...	195.00	731 25	1,225.50	34	28
Wisconsin	290.00	...	...	650.00	940.00	44	40	290 00	...	...	710 00	650.00	1,650.00	17	13
Minnesota	510.00	...	...	812.50	1,322.50	22	16	510.00	...	...	7.50	812 50	1,330.00	29	26
Iowa	595.00	...	...	650.00	1,245.00	24	18	595.00	...	...	5 00	650.00	1,250.00	31	27
Missouri	507.00	60.15	...	487.50	1,054.65	39	33	507 00	60 15	...	25 00	487.50	1,079.65	41	35
Southwestern:															
Arkansas	330.00	87.96	...	1,056.25	1,474 21	13	12	330.00	87.96	...	...	1,056.25	1,474.21	26	22
Louisiana	340.00	...	...	1,137.50	1,477.50	12	8	680.00	...	...	10.00	1,137 50	1,827.50	11	9
Oklahoma	485.00	...	1 00	1,056.25	1,542.25	9	6	485.00	...	1.00	...	1,056 25	1,542.25	20	18
Texas	337.00	118.23	...	650.00	1,105.23	36	35	337.00	118 23	...	11.00	650 00	1,116.23	40	38
West Central:															
North Dakota	700.00	...	125 00	812.50	1,637.50	5	1	700.00	...	125 00	50 00	812 50	1,687.50	13	12
South Dakota	192.50	...	565.00	812.50	1,570 00	8	5	192.50	...	565 00	10.00	812 50	1,580.00	19	16
Nebraska	321.00	295.49	...	812.50	1,428.99	17	22	321.00	295.49	...	20 00	812.50	1,448 99	27	30
Kansas	225.00	167.00	562 50	812.50	1,767 00	1	4	225.00	167.00	...	812.50	812.50	2,017.00	7	7
Mountain:															
Montana	166 25	269.93	...	975 00	1,411.18	18	21	166.25	269 93	...	270 00	975.00	1,681 18	15	21
Idaho	151.00	...	...	975.00	1,126.00	34	24	151 00	...	...	...	975.00	1,126.00	38	31
Wyoming	100.00	158.54	...	812.50	1,071.04	38	42	100.00	158.54	...	636.44	812 50	1,707.48	12	17
Colorado	65.00	158.54	1 00	975.00	1,199.54	28	29	65.00	158.54	1 00	1,390.00	975.00	2,589.54	1	2
New Mexico	184.50	...	...	975.00	1,159.50	32	19	92.25	...	...	975.00	975 00	2,042 25	6	5
Arizona	135.25	237.80	...	812.50	1,185 55	29	38	135.25	237.80	...	1,300.00	812.50	2,485 55	4	4
Utah	175.00	190.34	...	812.50	1,177 84	30	34	175.00	190.34	...	...	812 50	1,177 84	36	39
Nevada	76.95	378 13	305.05	731.25	1,491.38	11	25	76 95	378 13	...	305.05	731.25	1,491 38	24	33
Pacific:															
Washington	295 00	...	89.00	1,056.25	1,440.25	16	10	295 00	...	89.00	41.00	1,056 25	1,481.25	25	20
Oregon	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
California	162.00	...	152 00	731.25	1,045 25	40	28	162 00	...	152.00	1,510.00	731 25	2,555.25	2	1

counterpart. Consequently, in States where diesel fuel is taxed at the same rate as gasoline and there are no other special taxes on diesel vehicles, the annual saving to the operator in taxes paid is substantial. In the example used in this study (comparison of vehicles Nos. 8 and 9), the saving amounts to approximately $270 per year.

Distribution of vehicles.—Only 2 percent of our vehicles are trucks of 40,000 pounds or more capacity. The taxes paid by all classes of vehicles, including heavy trucks, should constantly be subject to review and adjustment, but from a practical standpoint the bulk of the revenue for highways must inevitably come from the automobiles and lighter trucks that constitute the vast majority of vehicles.

(Table 12 and figures 10-21 follow.)

Table 12.—Road-user and personal-property taxes on heavy combinations (diesel-powered)

State	Private operation							Contract carrier							
	Registration fee	Property tax	Other taxes and fees	Diesel fuel tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Diesel fuel tax	Total	Rank of State	
						Total fees and taxes	Total, excl. prop. tax							Total fees and taxes	Total, excl. prop. tax
FOUR-AXLE TRACTOR-SEMITRAILER COMBINATION, 50,000 POUNDS GROSS VEHICLE WEIGHT (NO. 9)															
New England:															
Maine	$355.00	$249.09	...	$649.98	$1,254.07	14	20	$355.00	$249.09	...	$10.00	$649.98	$1,264.07	25	30
New Hampshire	300.00	155.32	...	541.65	996.97	30	27	300.00	155.32	...	2.00	541.65	998.97	38	35
Vermont	1,065.00	...	...	...	1,065.00	24	13	1,065.00	...	...	...	...	1,065.00	34	25
Massachusetts	152.00	345.16	...	541.65	1,038.81	27	40	152.00	345.16	...	5.00	541.65	1,043.81	36	42
Rhode Island	152.00	406.00	...	433.32	991.32	31	44	152.00	406.00	...	5.00	433.32	996.32	39	44
Connecticut	250.00	394.86	...	433.32	1,078.18	22	42	250.00	394.86	...	5.00	433.32	1,083.18	32	43
Middle Atlantic:															
New York	220.50	...	$747.50	649.98	1,617.98	1	1	220.50	...	...	747.50	649.98	1,617.98	10	6
New Jersey	260.00	...	2.00	324.99	586.99	45	43	260.00	...	$2.00	...	324.99	586.99	45	45
Pennsylvania	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Delaware	190.00	...	...	541.65	731.65	43	35	190.00	...	...	...	541.65	731.65	44	40
Maryland	210.00	55.00	...	541.65	806.65	41	32	210.00	55.00	...	...	541.65	806.65	43	37
District of Columbia	195.00	231.00	...	541.65	967.65	34	34	195.00	231.00	...	...	541.65	967.65	40	39
West Virginia	317.00	431.45	...	541.65	1,290.10	13	24	317.00	431.45	...	47.00	541.65	1,337.10	23	33
Southeastern:															
Virginia	300.00	231.44	...	649.98	1,181.42	19	23	300.00	231.44	...	1,000.00	649.98	2,181.42	4	3
North Carolina	400.00	216.46	...	758.31	1,374.77	8	10	700.00	216.46	...	...	758.31	1,674.77	9	11
South Carolina	282.00	311.00	...	758.31	1,351.31	11	15	282.00	311.00	...	400.00	758.31	1,751.31	6	13
Georgia	170.00	244.86	...	649.98	1,064.84	25	30	337.50	244.86	...	25.00	649.98	1,257.34	26	31
Florida	230.00	...	.50	758.31	988.81	32	21	416.25	...	.50	325.50	758.31	1,500.56	12	10
Kentucky	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Tennessee	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Alabama	75.00	155.24	1.00	649.98	881.22	37	36	75.00	155.24	1.00	488.50	649.98	1,369.72	22	20
Mississippi	333.00	171.32	...	866.64	1,370.96	9	7	654.00	171.32	...	13.00	866.64	1,704.96	8	8
East Central:															
Ohio	376.80	...	...	433.32	810.12	40	31	376.80	...	...	30.00	433.32	840.12	42	36
Indiana	265.00	603.82	.25	433.32	1,302.39	12	39	265.00	693.82	.25	24.00	433.32	1,326.39	24	41
Illinois	789.00	93.57	...	541.65	1,424.22	5	5	789.00	93.57	...	...	541.65	1,424.22	18	17
Michigan	323.75	...	...	649.98	973.73	33	22	323.75	...	...	195.00	649.98	1,168.73	30	22
Wisconsin	290.00	...	...	433.32	723.32	44	37	290.00	...	...	710.00	433.32	1,433.32	17	14
Minnesota	510.00	...	...	541.65	1,051.65	26	14	510.00	...	...	7.50	541.65	1,059.15	35	26
Iowa	595.00	...	...	433.32	1,028.32	28	18	595.00	...	...	5.00	433.32	1,033.32	37	29
Missouri	507.00	60.15	...	324.99	892.14	36	29	507.00	60.15	...	25.00	324.99	917.14	41	34
Southwestern:															
Arkansas	330.00	87.96	...	704.15	1,122.11	20	16	330.00	87.96	...	...	704.15	1,122.11	31	28
Louisiana	340.00	...	...	758.31	1,098.31	21	12	680.00	...	...	10.00	758.31	1,448.31	15	12
Oklahoma	485.00	...	1.00	704.15	1,190.15	16	9	485.00	...	1.00	...	704.15	1,190.15	29	21
Texas	382.50	152.39	...	649.98	1,184.87	18	17	382.50	152.39	...	11.00	649.98	1,195.87	28	27
West Central:															
North Dakota	700.00	...	125.00	541.65	1,366.65	10	4	700.00	...	125.00	50.00	541.65	1,416.65	21	16
South Dakota	242.50	...	630.00	541.65	1,414.15	6	3	242.50	...	630.00	10.00	541.65	1,424.15	19	15
Nebraska	321.00	282.11	800.00	...	1,403.11	7	11	321.00	282.11	800.00	20.00	...	1,423.11	20	23
Kansas	225.00	217.00	562.50	541.65	1,546.15	2	6	225.00	217.00	...	812.50	541.65	1,796.15	5	7
Mountain:															
Montana	186.25	352.98	...	649.98	1,189.21	17	28	186.25	352.98	...	260.00	649.98	1,449.21	14	24
Idaho	78.00	...	1,446.90	...	1,524.90	3	2	78.00	...	...	1,446.90	...	1,524.90	11	9
Wyoming	110.00	207.09	...	433.32	750.41	42	45	110.00	207.09	...	685.60	433.32	1,436.01	16	19
Colorado	65.00	207.09	1.00	649.98	923.07	35	38	65.00	207.09	1.00	1,330.00	649.98	2,253.07	2	2
New Mexico	204.00	...	...	649.98	853.98	39	26	102.00	...	...	975.00	649.98	1,726.98	7	5
Arizona	145.75	310.60	...	541.65	998.00	29	41	145.75	310.60	...	1,250.00	541.65	2,248.00	3	4
Utah	195.00	339.76	...	541.65	1,076.41	23	33	195.00	339.76	...	...	541.65	1,076.41	33	38
Nevada	83.70	493.93	326.75	595.82	1,500.20	4	19	83.70	493.93	...	326.75	595.82	1,500.20	13	32
Pacific:															
Washington	370.00	...	122.25	704.15	1,196.40	15	8	370.00	...	122.25	42.00	704.15	1,238.40	27	18
Oregon	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
California	172.00	...	198.00	487.49	857.49	38	25	172.00	...	198.00	1,446.67	487.49	2,304.16	1	1
FIVE-AXLE TRACTOR-SEMITRAILER COMBINATION, 64,000 POUNDS GROSS VEHICLE WEIGHT (NO. 10)															
Maryland	$235.00	$60.00	...	$851.05	$1,146.05	20	18	$235.00	$60.00	...	...	$851.05	$1,146.05	20	19
District of Columbia	365.00	314.10	...	851.05	1,530.15	13	14	365.00	314.10	...	...	851.05	1,530.15	19	17
South Carolina	347.00	427.00	...	1,191.47	1,965.47	5	6	347.00	427.00	...	$400.00	1,191.47	2,365.47	6	11
Florida	295.50	...	$.50	1,191.47	1,487.47	15	9	601.50	...	$.50	400.50	1,191.47	2,193.97	11	7
Indiana	300.00	777.33	.25	680.84	1,758.42	8	19	300.00	777.33	.25	24.00	680.84	1,782.42	16	20
Michigan	479.75	...	...	1,021.26	1,501.01	14	8	479.75	...	...	320.00	1,021.26	1,821.01	15	12
Arkansas	455.00	87.96	...	1,106.37	1,649.33	10	5	455.00	87.96	...	...	1,106.37	1,649.33	18	14
Louisiana	400.00	...	...	1,191.47	1,591.47	11	4	800.00	...	...	10.00	1,191.47	2,001.47	13	8
South Dakota	642.50	...	730.00	851.05	2,223.55	2	2	642.50	...	730.00	10.00	851.05	2,233.55	8	6
Nebraska	401.00	651.95	1,000.00	...	2,052.95	4	10	401.00	651.95	1,000.00	20.00	...	2,072.95	12	16
Montana	336.25	469.99	...	1,021.26	1,827.50	7	11	336.25	469.99	...	400.00	1,021.26	2,227.50	9	13
Idaho			2,926.40	...	2,926.40	1	1	...	...	...	2,926.40	...	2,926.40	4	4
Wyoming	180.00	287.84	...	680.84	1,148.68	19	20	180.00	287.84	...	1,107.00	680.84	2,255.68	7	10
Colorado	100.00	287.84	1.00	1,021.26	1,410.10	16	16	100.00	287.84	1.00	2,170.00	1,021.26	3,580.10	2	2
New Mexico	304.50	...	...	1,021.26	1,325.76	18	13	152.25	...	...	1,200.00	1,021.26	2,373.51	5	5
Arizona	258.75	431.80	...	851.05	1,541.60	12	17	258.75	431.80	...	1,950.00	851.05	3,491.60	3	3
Utah	350.00	476.16	...	851.05	1,677.21	9	15	350.00	476.16	...	...	851.05	1,677.21	17	18
Nevada	113.40	686.51	485.75	936.16	2,221.82	3	7	113.40	686.51	...	487.75	936.16	2,221.82	10	15
Washington	662.50	...	171.75	1,106.37	1,940.62	6	3	662.50	...	171.75	47.00	1,106.37	1,987.62	14	9
Oregon	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
California	302.00	...	270.00	765.95	1,337.95	17	12	302.00	...	270.00	2,243.34	769.95	3,581.29	1	1
SIX-AXLE TRUCK AND FULL TRAILER COMBINATION, 72,000 POUNDS GROSS VEHICLE WEIGHT (NO. 11)															
Ohio	$703.75	...	...	$837.20	$1,540.95	10	7	$703.75	...	...	$20.00	$837.20	$1,560.95	12	11
Louisiana	520.00	...	...	1,465.10	1,985.10	5	3	1,040.00	...	...	10.00	1,465.10	2,515.10	9	7
Montana	422.50	$530.11	...	1,255.80	2,208.41	4	5	422.50	$530.11	...	490.00	1,255.80	2,698.41	7	9
Idaho	...	...	$3,427.20	...	3,427.20	1	1	...	...	...	3,427.20	...	3,427.20	5	5
Wyoming	220.00	310.79	...	837.20	1,367.99	12	12	220.00	310.79	...	1,688.50	837.20	3,056.49	6	6
Colorado	107.50	310.79	1.00	1,255.80	1,675.09	8	10	107.50	310.79	$1.00	2,800.00	1,255.80	4,475.09	1	2
New Mexico	334.50	...	...	1,255.80	1,590.30	9	6	167.25	...	...	2,700.00	1,255.80	4,123.05	4	3
Arizona	277.00	466.20	...	1,046.50	1,789.70	7	11	277.00	466.20	...	2,400.00	1,046.50	4,189.70	3	4
Utah	445.00	362.70	...	1,046.50	1,854.20	6	9	445.00	362.70	...	...	1,046.50	1,854.20	11	12
Nevada	129.60	741.23	628.50	1,151.15	2,650.48	2	4	129.60	741.23	...	628.50	1,151.15	2,650.48	8	10
Washington	842.50	...	180.25	1,360.45	2,383.20	3	2	842.50	...	180.25	50.00	1,360.45	2,433.20	10	8
Oregon	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
California	302.00	...	293.00	941.85	1,536.85	11	8	302.00	...	293.00	2,783.33	941.85	4,320.18	2	1

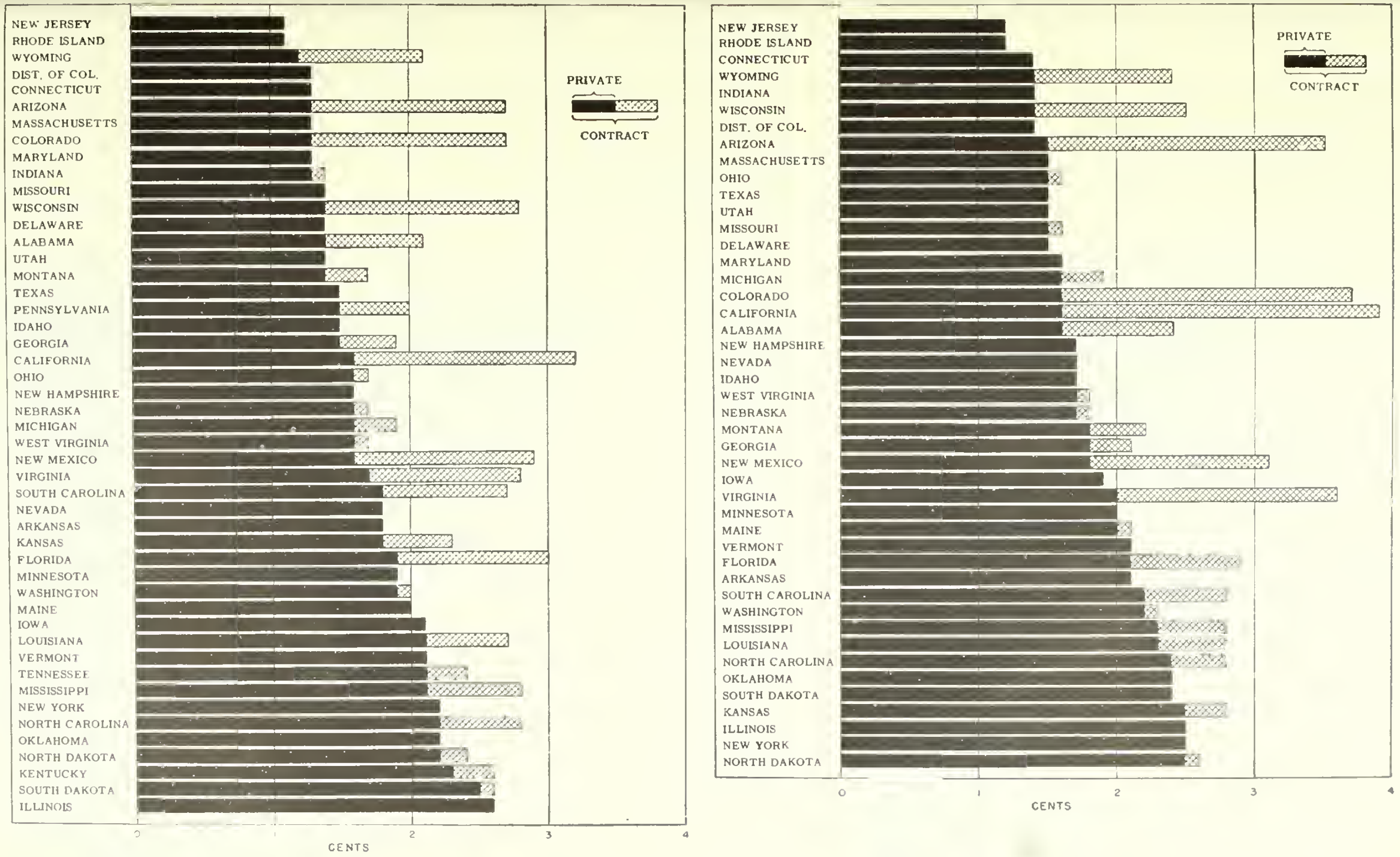
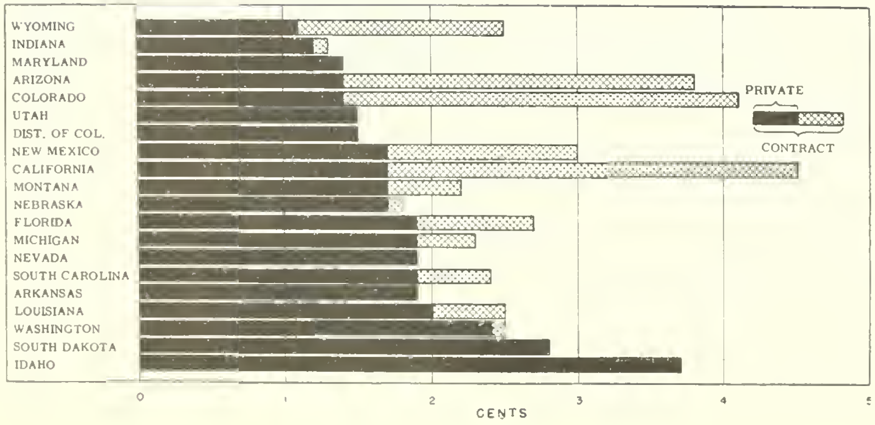


Figure 10.—Comparison of State road-user taxes (personal-property taxes not included), in cents per mile for private use and contract operation, for a 40,000-pound three-axle tractor-semitrailer (No. 7) (left) and a 50,000-pound gasoline-powered four-axle tractor-semitrailer (No. 8) (right).



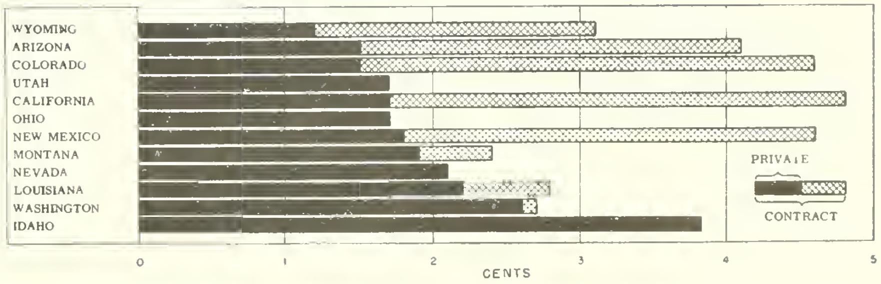


Figure 11.—Comparison of State road-user taxes (personal-property taxes not included) in cents per mile for private use and contract operation, for a 64,000-pound diesel-powered five-axle tractor-semitrailer (No. 10) (left) and a 72,000-pound diesel-powered six-axle combination, truck and full trailer (No. 11) (right).

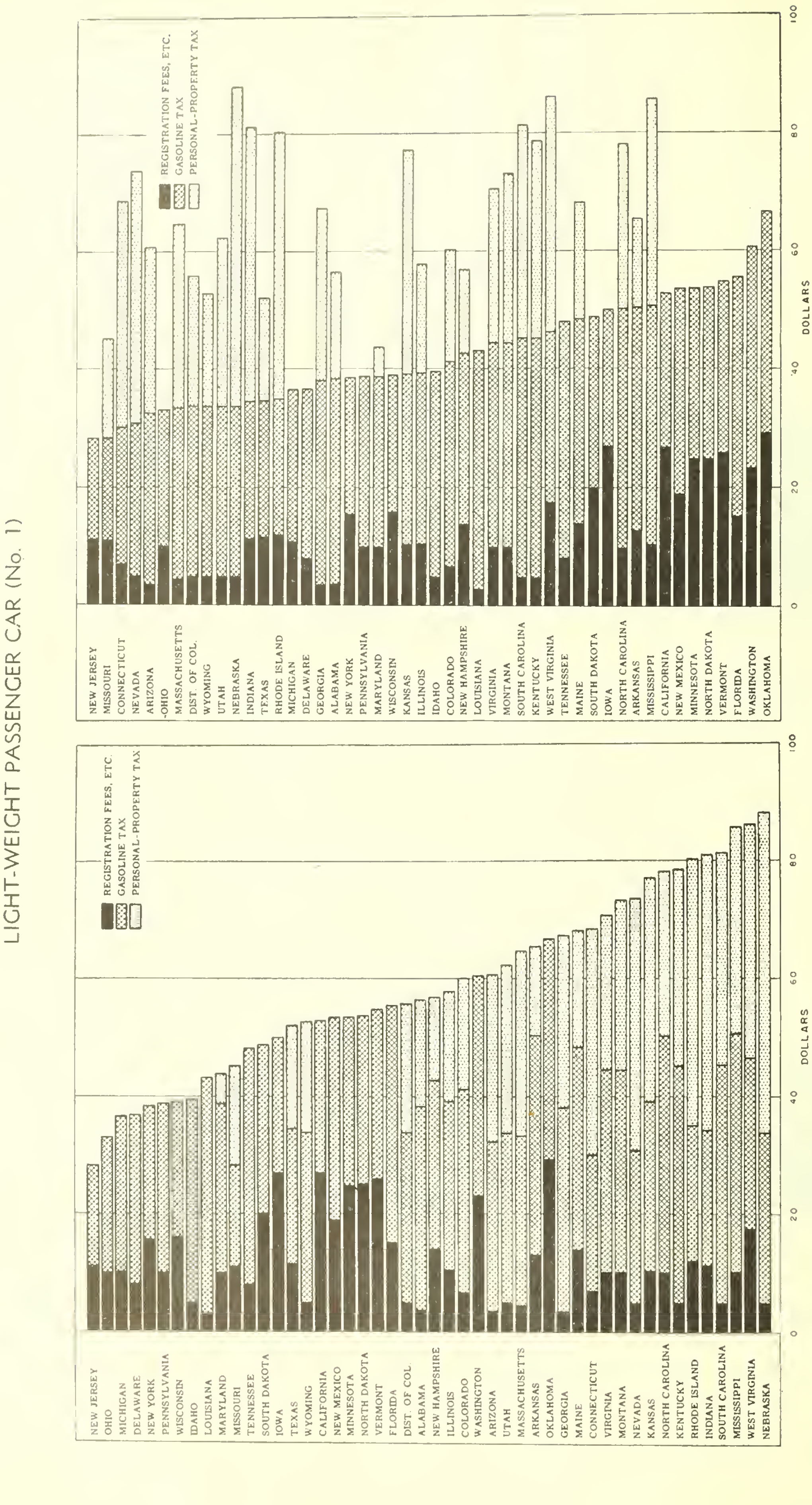


Figure 12.—State road-user and personal-property taxes on a light-weight passenger car (No. 1), ranked according to total taxes (left) and road-user taxes (right).

PICKUP TRUCK (No. 3)

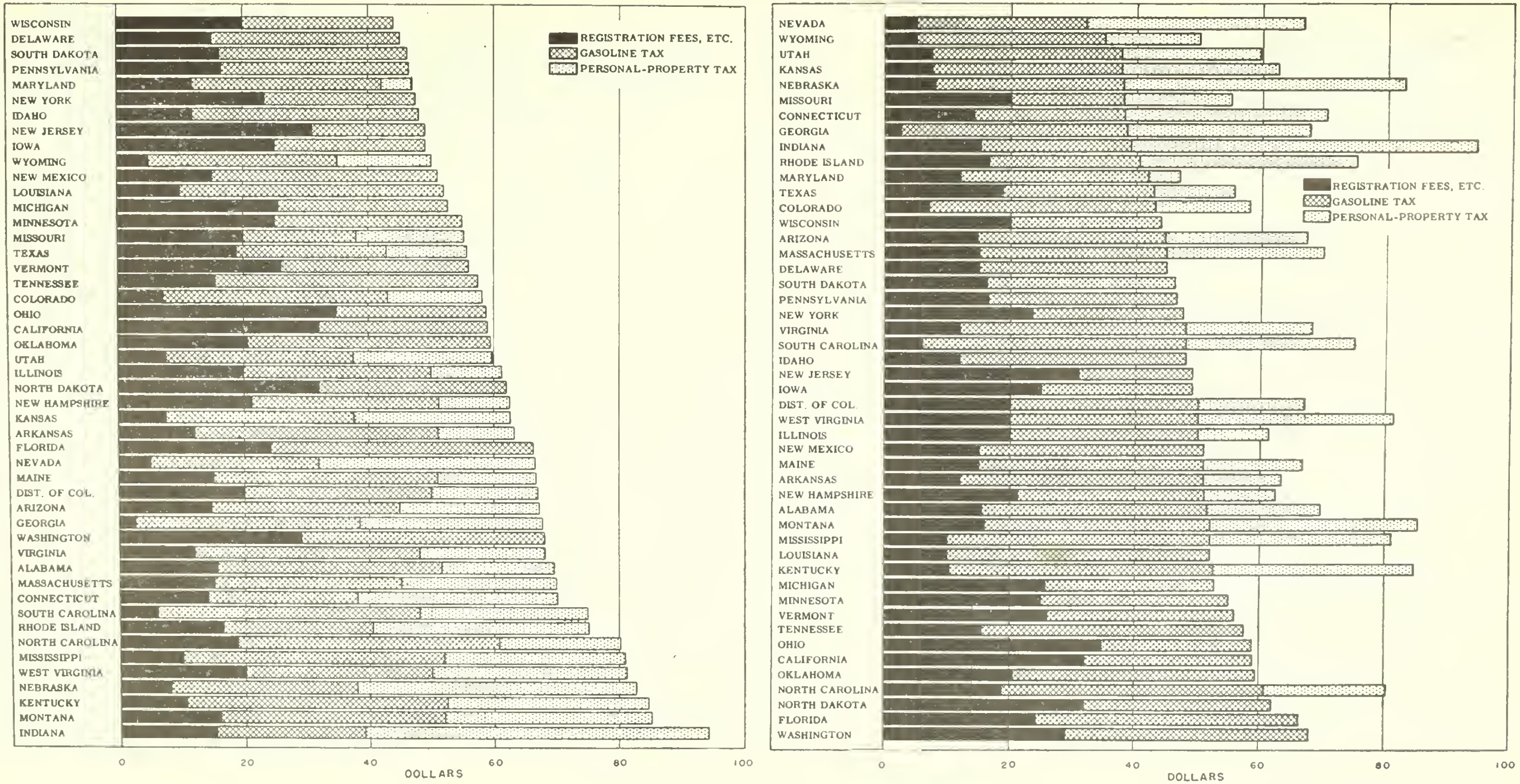


Figure 13.—State road-user and personal-property taxes on a pickup truck (No. 3) in private use, ranked according to total taxes (left) and road-user taxes (right).

12,500-POUND STAKE TRUCK (No. 4)

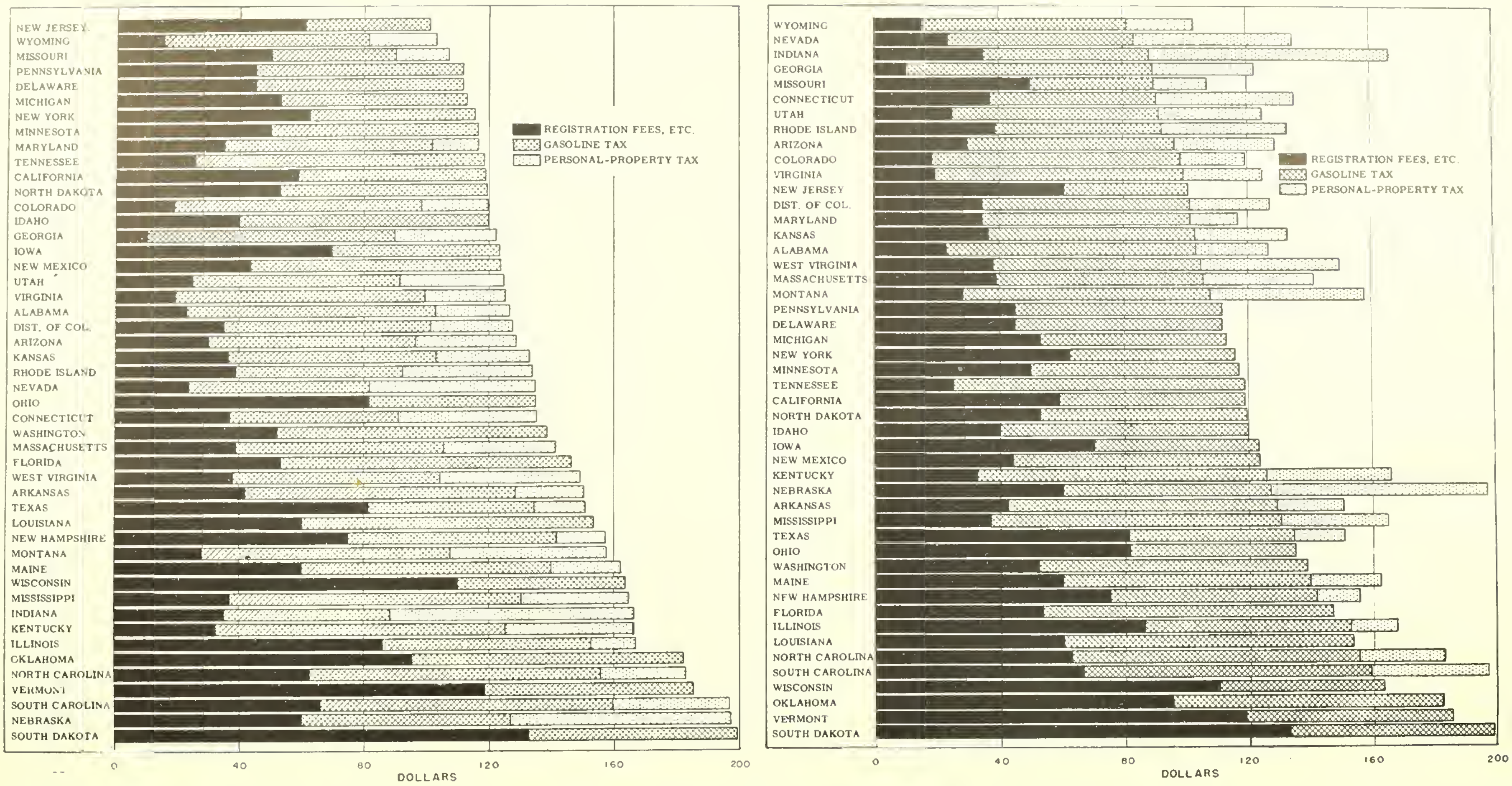


Fig. 14.—State road-user and personal-property taxes on a 12,500-pound stake truck (No. 4) in private use, ranked according to total taxes (left) and road-user taxes (right).

19,000-POUND VAN TRUCK (No. 5)

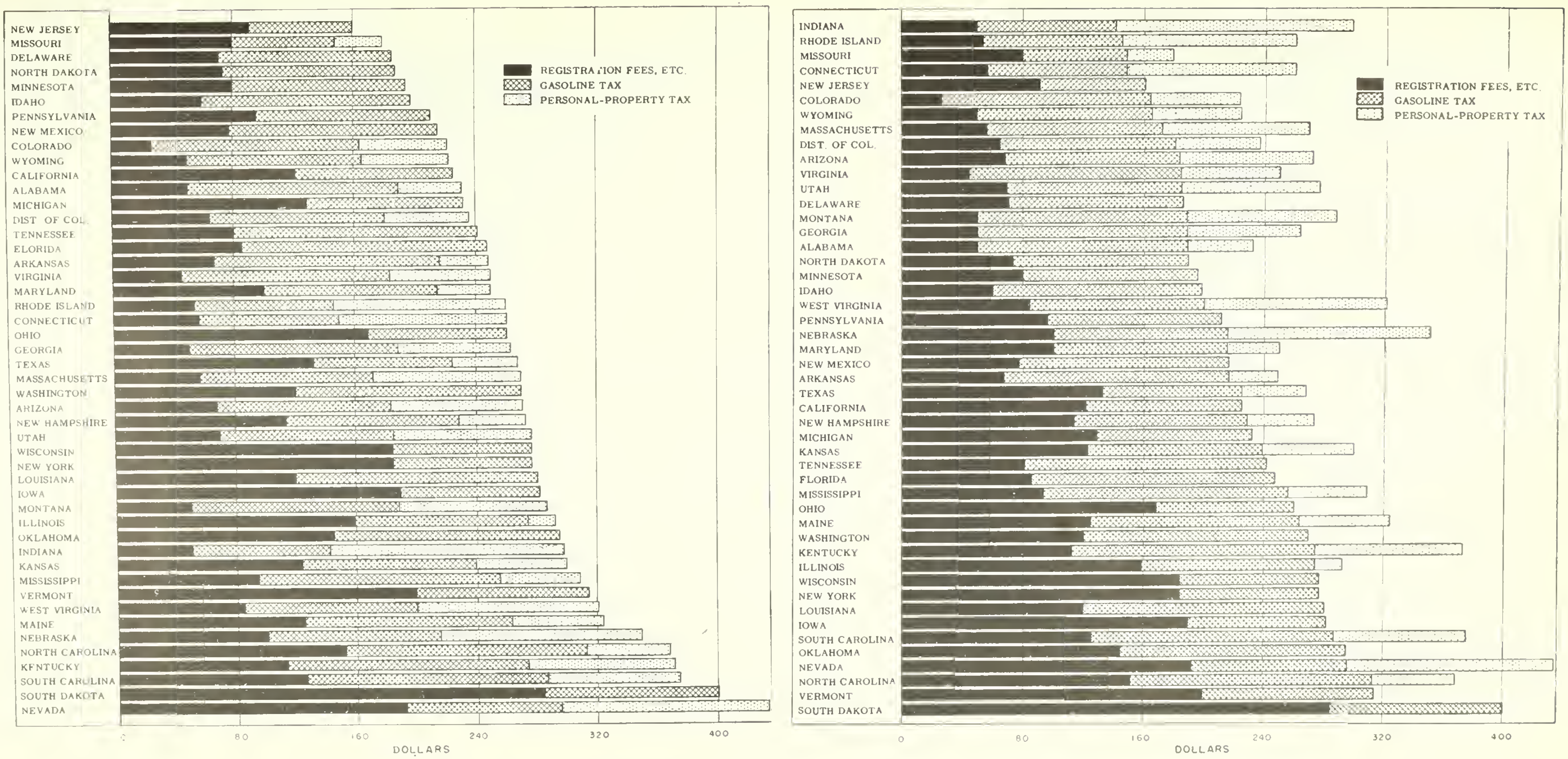


Figure 15.—State road-user and personal-property taxes on a 19,000-pound van truck (No. 5) in private use, ranked according to total taxes (left) and road-user taxes (right).

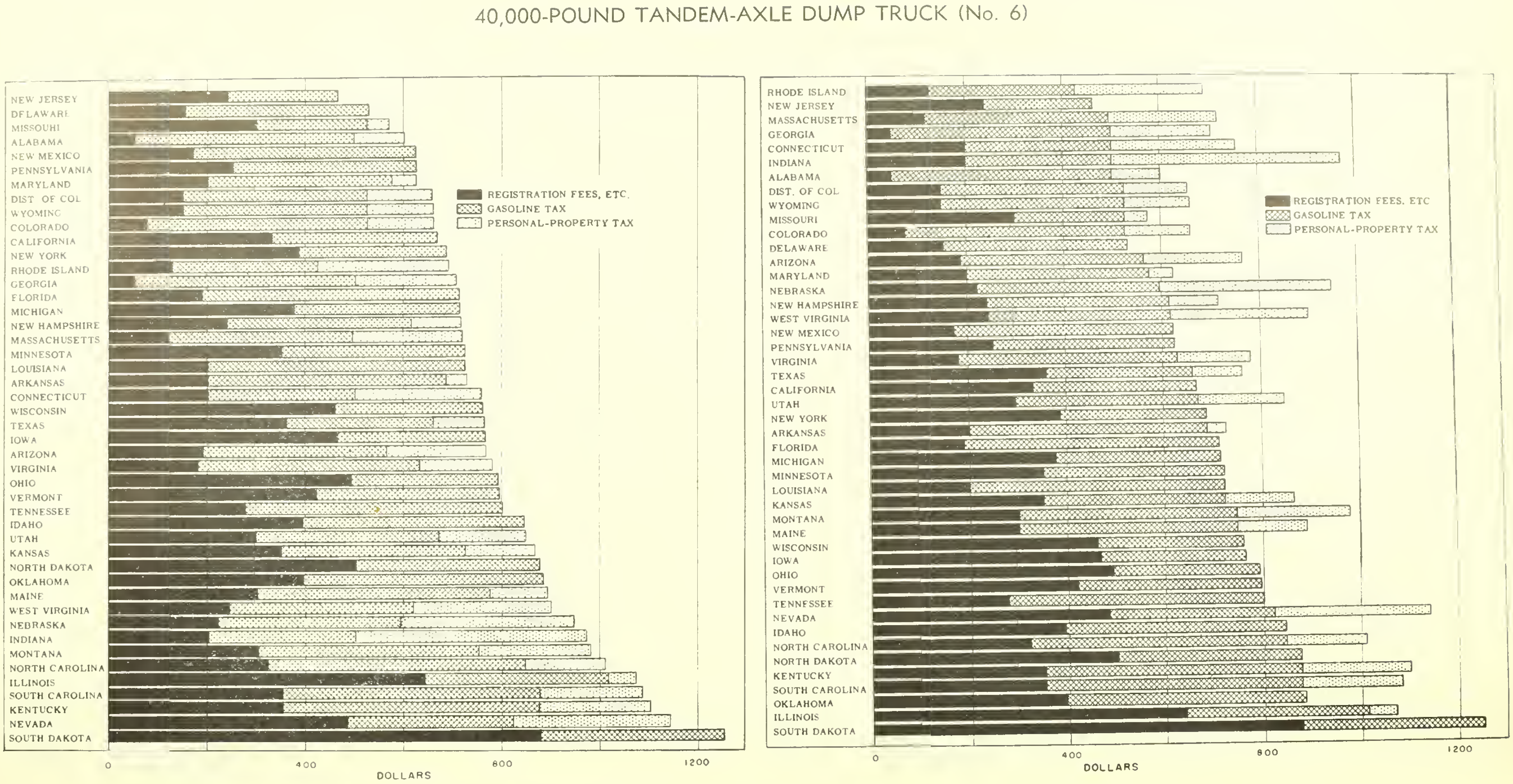

40,000-POUND TANDEM-AXLE DUMP TRUCK (No. 6)

Figure 16.—State road-user and personal-property taxes on a 40,000-pound tandem-axle dump truck (No. 6) in private use, ranked according to total taxes (left) *and road-user taxes* (right).

40,000-POUND THREE-AXLE TRACTOR-SEMITRAILER (No. 7)

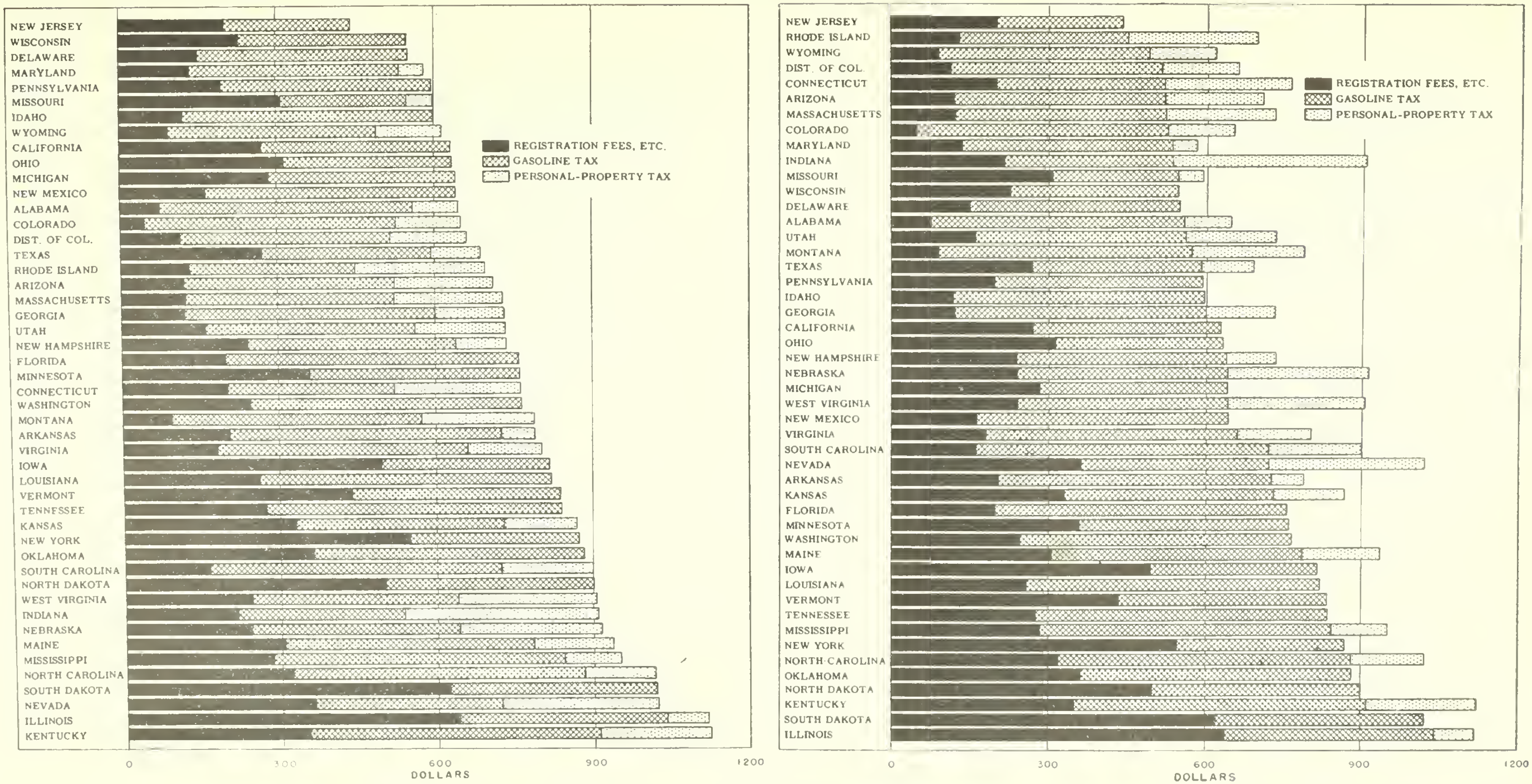


Figure 17.—State road-user and personal-property taxes on a 40,000-pound three-axle tractor-semitrailer combination (No. 7) in private use, ranked according to total taxes (left) and road-user taxes (right).

50,000-POUND GASOLINE-POWERED FOUR-AXLE TRACTOR-SEMITRAILER (No. 8)

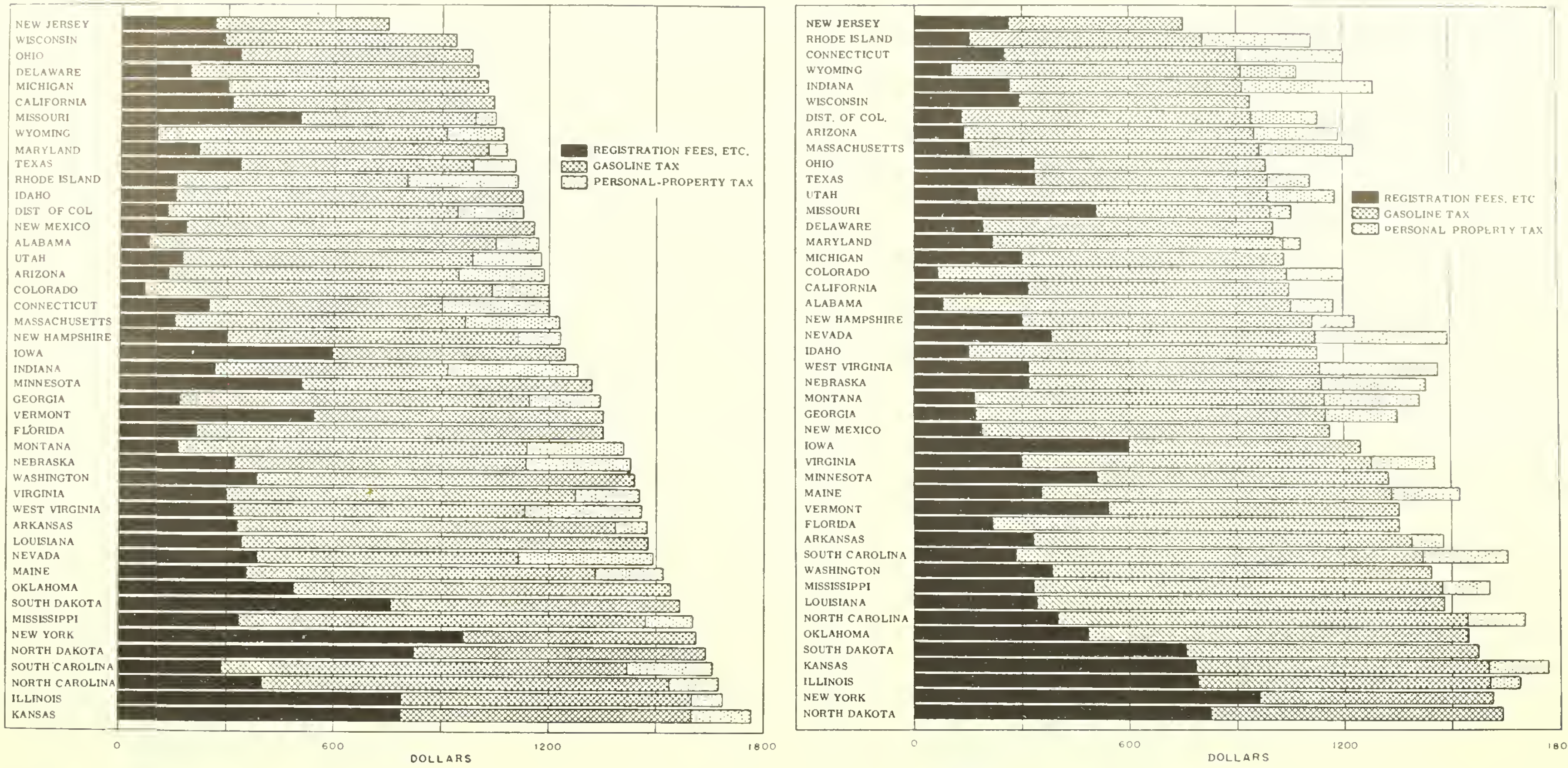


Figure 18.—State road-user and personal-property taxes on a 50,000-pound gasoline-powered four-axle tractor-semitrailer combination (No. 8) in private use, ranked according to total taxes (left) *and road-user taxes* (right).

50,000-POUND DIESEL-POWERED FOUR-AXLE TRACTOR-SEMITRAILER (No. 9)

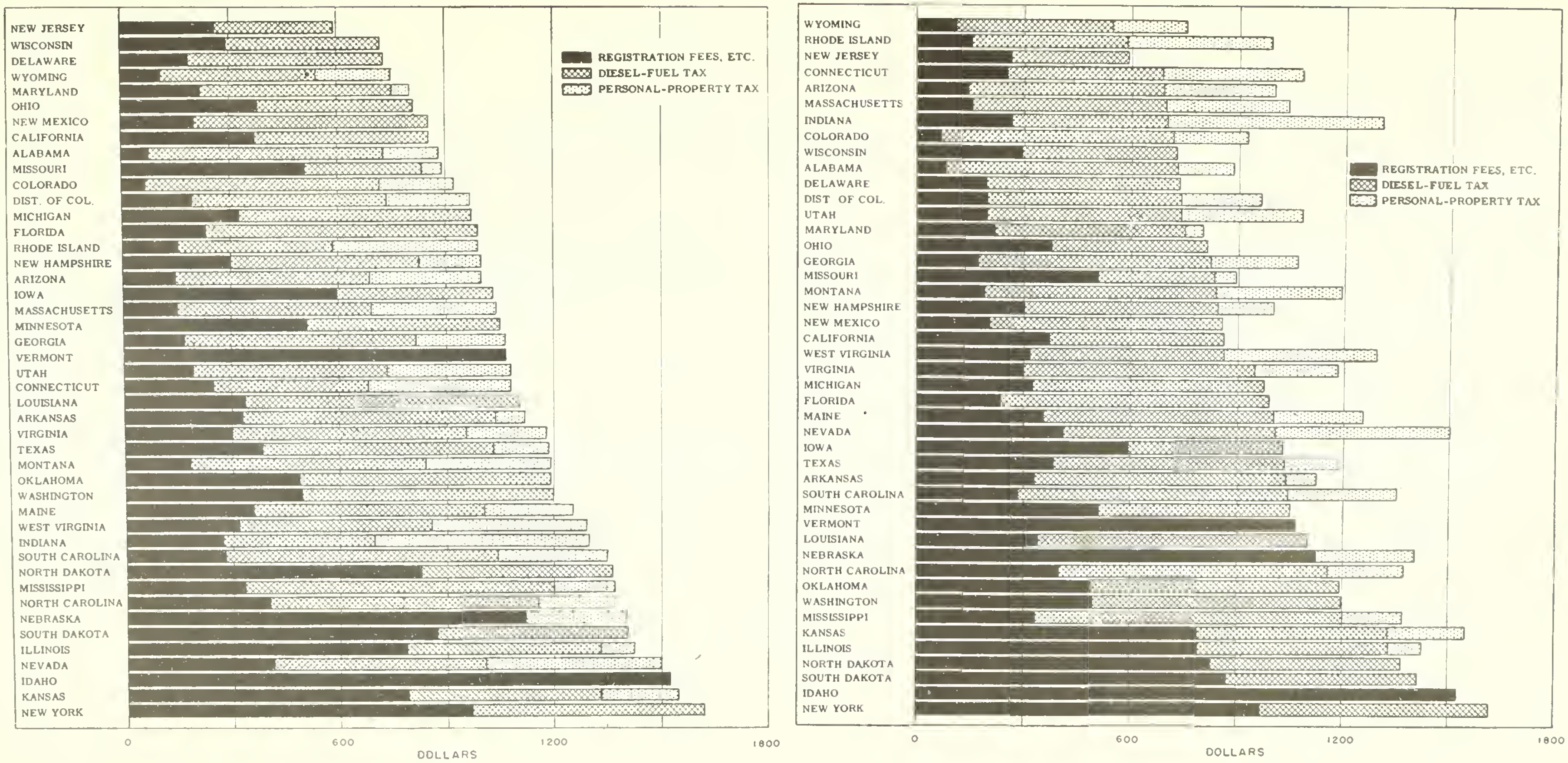


Figure 19.—State road-user and personal-property taxes on a 50,000-pound diesel-powered four-axle tractor-semitrailer combination (No. 9) in private use, ranked according to total taxes (left) and road-user taxes (right).

64,000-POUND DIESEL-POWERED FIVE-AXLE TRACTOR-SEMITRAILER (No. 10)

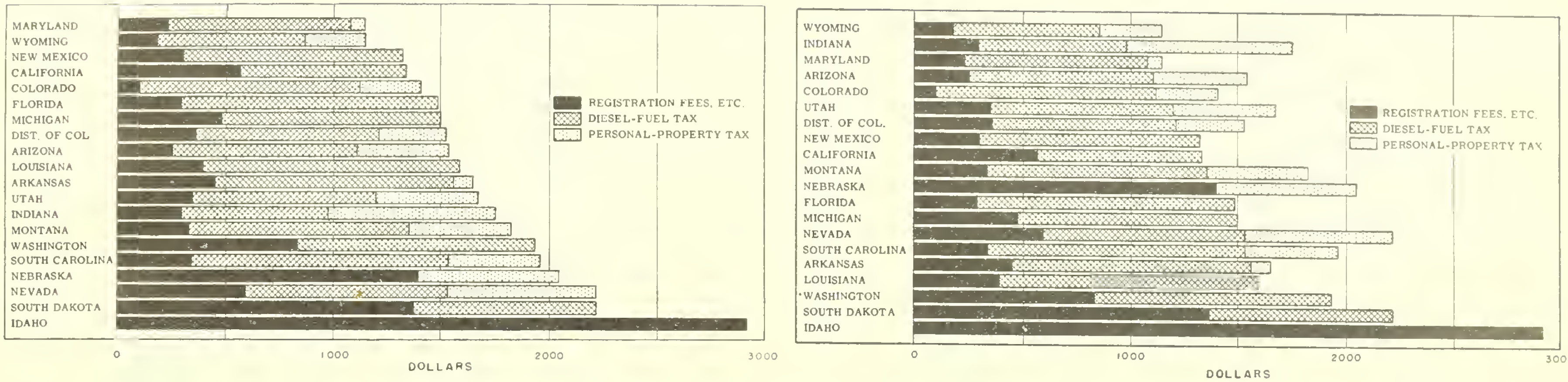


Figure 20.—State road-user and personal-property taxes on a 64,000 pound diesel-powered five-axle tractor-semitrailer combination (No. 10) in private use, ranked according to total taxes (left) *and road-user taxes* (right).

72,000-POUND DIESEL-POWERED SIX-AXLE TRUCK AND FULL TRAILER (No. 11)

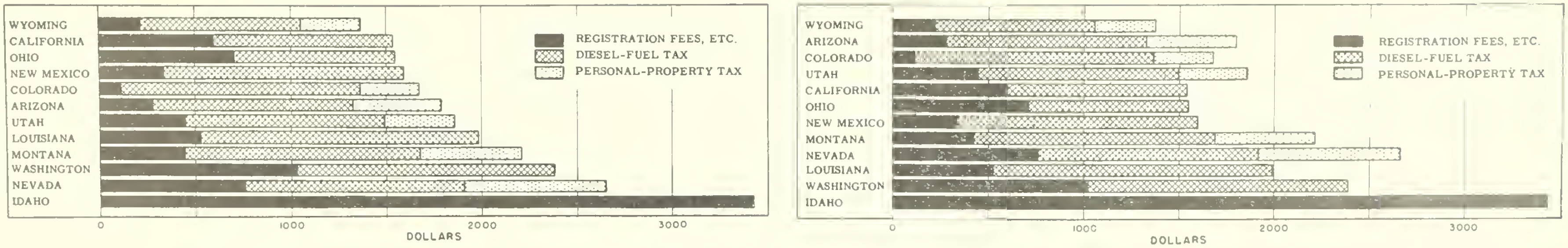


Figure 21.—State road-user and personal-property taxes on a 72,000-pound diesel-powered six-axle truck and full trailer combination (No. 11) in private use, ranked according to total taxes (left) *and road-user taxes* (right).

Plotting Aggregate Gradation Specifications for Bituminous Concrete

By JOHN B. DALHOUSE, Highway Engineer
North Carolina District Office
Bureau of Public Roads

Aggregate gradation specifications for bituminous concrete generally specify either the total percentages passing each sieve of a series or the percentages passing one sieve and retained on the next. Ready comparison of two dissimilar types has been considered impossible, largely because no practical means of plotting the second type for visual inspection has heretofore been developed.

In this article there is described a simple method for plotting the requirements of percentage passing and retained specifications, thereby permitting visualization of the nature of a specified gradation and comparison of it with the plotting of a total percentage passing gradation.

ALL SPECIFICATIONS for bituminous concrete set forth the gradation of materials to be used in the mix. Mineral aggregate gradation requirements in the various current specifications are of two general types. One specifies the total percentages passing each individual sieve in a given series, and the other specifies the percentages passing one sieve and retained on the next smaller sieve. Because of the dissimilarity between the two types, it has in the past generally been considered impossible to compare them by any ready means. An additional complication is the use of varying series of sieves in different specifications.

Some experienced engineers are able to determine, by reading of the specifications, just what kind of a mix will result from the requirements. To a great many engineers, however, the easiest means of visualizing the nature of a specified aggregate gradation is to plot it as a graph, preferably on semilogarithmic cross-section paper. A convenient form of aggregate chart is commonly used, with a percentage scale on the vertical axis and the sieve sizes in logarithmic scale on the horizontal axis.

The plotting is an easy operation for the percentage passing specifications, but has not heretofore been feasible by any simple method for specification requirements on a percentage passing and retained basis. The purpose of this report is to describe a method whereby this latter type of specification can be plotted.

Demonstration of Method

To demonstrate the method, the passing and retained aggregate gradation specification shown in table 1 is used in this report. It is to be understood that this is not considered as a typical or recommended gradation, but merely as a working example. Derivation of the plotting points is shown in table 2, and the curves drawn therefrom appear in figures 1 and 2.

The percentages of material passing and retained are given in table 1 as ranges, in the usual manner. These figures are, of course, minimum and maximum amounts. The first step in the procedure is to calculate from table 1 the cumulative percentages of material passing each sieve, using first the minimum limits throughout and second, the maximum limits starting with the smallest size sieve in the sieve series in each case. These cumulative series are entered respectively in columns 1 and 2 of table 2. For example, working upward from the bottom of table 1, the *minimum* cumulative amount passing the No. 200 sieve is 2 percent, that passing the No. 80 sieve is 2 + 7 = 9 percent, and so on (see column 1 of table 2). The *maximum* cumulative amount passing the No. 200 sieve is 8 percent, that passing the No. 80 sieve is 8 + 12 = 20 percent, and so on (see column 2 of table 2).

Table 1.—Assumed aggregate gradation requirements

Sieve size		Percentage of material
Passing	Retained on	
......	1 inch	0
1 inch	½ inch	25–45
½ inch	No. 4	10–25
No. 4	No. 8	6–15
No. 8	No. 16	6–9
No. 16	No. 40	8–13
No. 40	No. 80	7–13
No. 80	No. 200	7–12
No. 200	......	2–8

The minimum cumulative percentage passing values thus calculated are plotted in figure 1 as curve 1; the maximum values as curve 2. As indicated in table 2, column 2, certain of the calculated percentages may be greater than 100 percent and are disregarded in the plotting since they are fictional values. The two curves define the limits of the intended grading band in the fine sizes but they do not define them in the coarse sizes.

The second step in the procedure is to calculate from table 1 the cumulative percentages of material retained on each sieve, using first the minimum limits throughout and second, the maximum limits, starting with the maximum size sieve in the sieve series in each case. Enter these figures in columns 1R and 2R of table 2. Thus, working downward from the top of table 1, the *minimum* cumulative amount retained on the 1-inch sieve is 0, that retained on the ½-inch sieve is 25 percent, that retained on the No. 4 sieve is 25 + 10 = 35 percent, and so on (see column 1R of table 2). The *maximum* cumulative amount retained on the 1-inch sieve is 0, that retained on the

Table 2.—Cumulative percentages of aggregate gradation, and derived specification on a percentage passing basis

Sieve size	Cumulative percentage passing, by simple addition		Cumulative percentage retained		Cumulative percentage passing, by substraction		Derived specification on a percentage passing basis [1] (5)
	Minimum (1)	Maximum (2)	Minimum (1R)	Maximum (2R)	Maximum (3)	Minimum (4)	
1 inch	71	[2] 140	0	0	100	100	100
½ inch	46	95	25	45	75	55	55–75
No. 4	36	70	35	70	65	30	36–65
No. 8	30	55	41	85	59	15	30–55
No. 16	24	46	47	94	53	6	24–46
No. 40	16	33	55	[2] 107	45	[2] 7	16–33
No. 80	9	20	62	[2] 120	38	[2] −20	9–20
No. 200	2	8	69	[2] 132	31	[2] −32	2–8

[1] The greater of the two minimum values, columns 1 and 4, and the lesser of the two maximum values, columns 2 and 3.

[2] Fictional values.

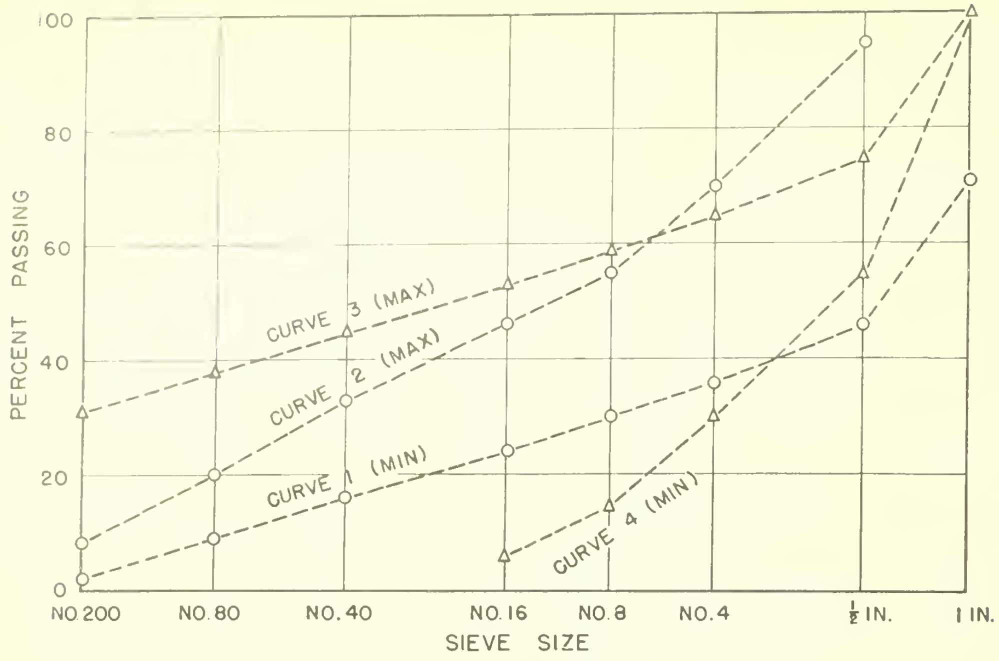


Figure 1.—Limiting curves of grading.

½-inch sieve is 45 percent, that retained on the No. 4 sieve is $45 + 25 = 70$ percent, and so on (see column 2R of table 2).

In any grading analysis, when the percentage retained on any particular sieve is known, the percentage passing the same sieve can be determined simply by subtracting the percentage retained from 100. Thus, in table 2, columns 3 and 4 are derived from columns 1R and 2R. The values exceeding 100 percent in column 2R are fictional and, therefore, the negative values in column 4 are also fictional and are disregarded in plotting.

The maximum cumulative percentage passing values thus calculated (column 3, table 2) are plotted in figure 1 as curve 3; the minimum values (column 4, table 2) as curve 4. These two curves define the limits of the intended grading band in the coarse sizes but they do not define them in the fine sizes.

The third step is to select the lower values of curves 2 and 3 and connect these values by a solid line, as illustrated in figure 2. Similarly, the higher values of curves 1 and 4 are selected and connected by a solid line. These curves define the limits of the intended grading band.

As indicated in column 5 of table 2, an equivalent gradation on a percentage passing basis may be derived from the values computed in columns 1, 2, 3, and 4. The lowest maximum values (columns 2 and 3) and the highest minimum values (columns 1 and 4) are used. When plotted on an aggregate chart, these percentage passing values enclose an area identical in shape and size with that representing the passing and retained specification from which they are derived.

The grading band, so drawn, will enclose all possible gradings that fall within the limits of the passing and retained grading specification in table 1, and the derived percentage passing grading, in column 5 of table 2.

Control for each Sieve

The two specifications are not identical in their control, however, because of another factor. That is the requirement in the passing and retained specification, table 1, that for each fraction defined, there is a definite control of the amount of that which passes any particular sieve which must be retained on the next smaller sieve in the named series. This has the effect of placing maximum and minimum limits on the slope of each segment of any specific grading curve that might be drawn within the borders of the grading band. These maximum and minimum slopes vary as the specific grading curve passes from zone to zone but they are definitely fixed, as follows:

1. Minimum slopes are fixed by curves 1 and 3 which are parallel to each other throughout (see fig. 2). Therefore, each of the solid lines in the envelope curve that represents a segment of either curve 1 or 3 establishes the minimum slope, under the passing and retained specification, for the zone in which it occurs.

2. Maximum slopes are fixed by curves 2 and 4 which also are parallel to each other throughout, including the fictional segments not shown. Therefore, each of the solid lines in the envelope curve that represent a segment of either curve 2 or 4 establishes the maximum slope for its zone.

3. In the transition zone B, no segment of curves 1 or 4 was used as a part of the envelope curve. Likewise in zone C, no segment of curves 2 or 3 was used as a part of the envelope curve. In these zones, proper control may be obtained by using the dashed segment of curve 1 to set the minimum permissible slope in zone B and the dashed segment of curve 4 to set the maximum permissible slope in that zone. Similarly, the dashed segments of curves 3 and 2 establish the minimum and maximum slopes respectively in zone C. It might appear that this is not necessary and that it would be more practical to utilize the solid lines connecting Y with Z and V with X for the slope control, but this is not correct as may be seen by a close examination of the specification requirements.

It is evident, from the preceding dis-

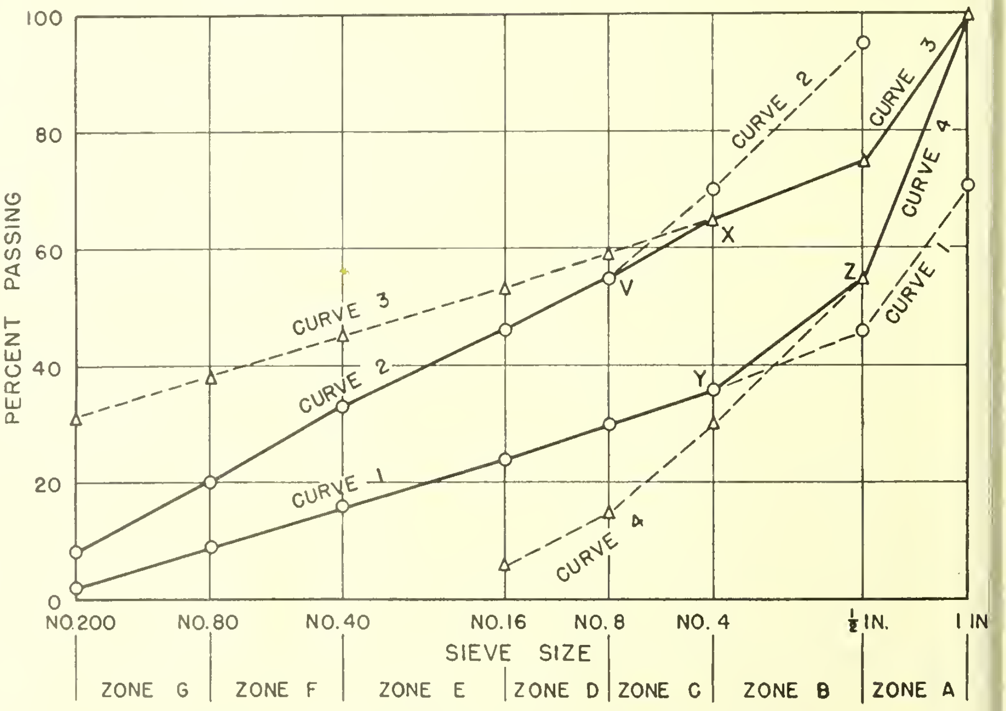


Figure 2.—Grading band (enclosed area between solid lines).

cussion, that the number of different grading curves that could be drawn within the boundaries of a grading band, such as the one illustrated in figure 2, is greatly reduced by the slope limitations that are imposed in each zone by the passing and retained type of specification. There is, of course, no such reduction in the case of the ordinary type of percentage passing specification. As a result, unless some slope limit clause is included in the grading requirements of the percentage passing type, a great many more grading curves of this type can be included in the grading band under discussion than will meet the requirements of the passing and retained specification on which this grading band is based.

The heavy dot-dash line within the grading band limits of figure 3 represents an individual grading or job-formula curve that conforms to both the fractional limits and slope limits of the illustrative passing and retained grading of table 1. Actually, this curve is also fairly representative of job formulas that are generally developed under the percentage passing type of specification. Any curve within the grading band, such as the one illustrated in figure 3, will meet the requirements of the passing and retained specification provided the slopes within the grading zones are within the minimum and maximum slopes indicated by the dotted lines.

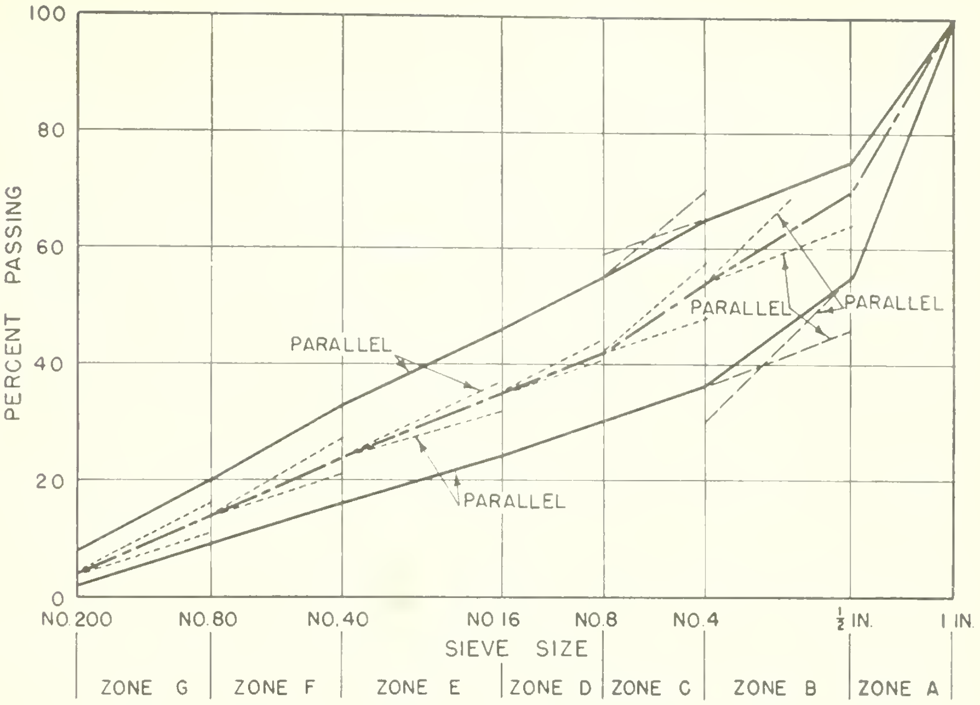


Figure 3.—Example of a grading that will pass both types of specification.

Skip Grading

The simple requirement that the job-formula curves must fall within the grading limits of the percentage passing specification in column 5 of table 2, or the grading band defined by the solid lines of figures 2 and 3, would permit another type of job formula characterized by one or more "skips" in the grading curve—that is, one in which, in one or more zones of the curve, all material passing a certain sieve will also pass the next smaller sieve. Such a job-formula curve is shown by the heavy dot-dash line in figure 4. It indicates a skip grading in that all of the material that passes the No. 4 sieve also passes the No. 8. Some engineers permit skip gradings to be used on the ground that they have certain definite advantages. Many others, however, do not permit their use. In such cases, in order to provide for the exclusion of skip gradings, it may be necessary to have controls other than the simple grading band limits for the percentage passing specification. Skip gradings are automatically eliminated by most specifications of the passing and retained type. They are readily prevented in percentage passing specifications by requiring that there be a certain minimum amount of material between any two consecutive sieve sizes.

For example, assume that the percentage passing specification in column 5 of table 2 had an additional requirement that the fraction retained between any two consecutive sieves shall not be less than 4 percent of the total. Then any curve within the grading band in figure 4 would represent a gradation meeting the specifications provided the slopes within the grading zones are greater than this minimum 4 percent shown by the dotted lines.

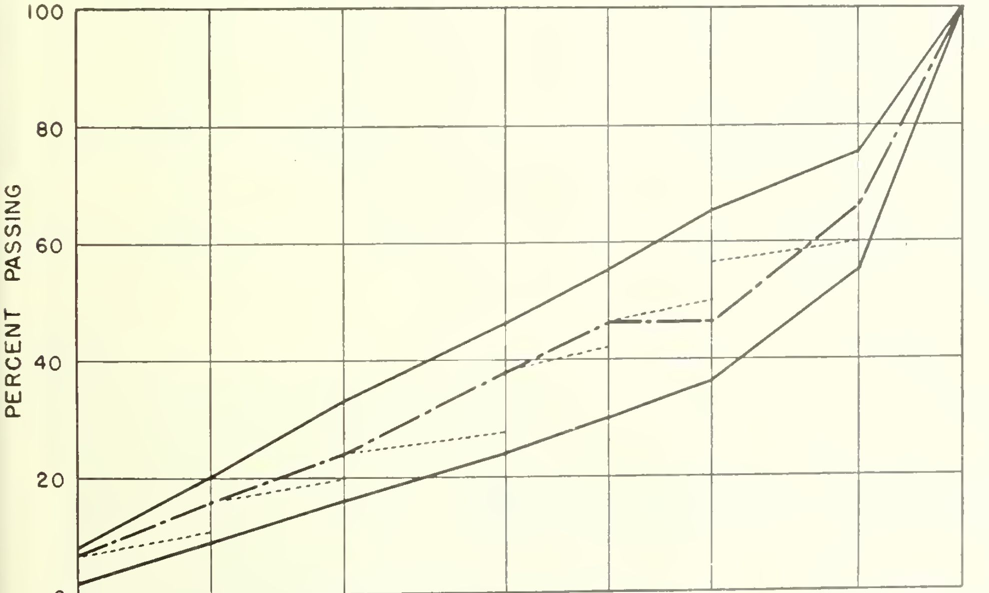


Figure 4.—Illustration of a skip grading.

Other Controls

As noted, a skip grading, represented by a zero slope illustrated in zone C in figure 4, is considered by many engineers to be an undesirable extreme. This is debatable, but there is no doubt as to the undesirability of the opposite extreme of excess material in one size which would be reflected by a very steep slope in one or more of the zones. With the grading band limits as wide as those in figure 4, it would be possible to have as much as 39 percent of the aggregate between two sieves (zone B). The latter extreme can be partially avoided by reducing the percentage range in the specifications for all or some of the sieves. However, actual experience with the passing type of specification indicates that the extreme conditions discussed above do not generally constitute a serious problem.

Some passing and retained specifications with relatively wide controlling limits have an additional requirement such as, for instance, that the total retained on the No. 10 sieve shall be 45–65 percent (equivalent to percentage passing of 35–55 percent). This type of control may be desirable, particularly on the No. 8 or No. 10 sieve, but it is misleading to set the envelope limits to appear to give a wider range.

In other cases, slopes are drastically limited by such means as setting very narrow ranges on certain sieves, as illustrated in zone D of figure 3, where the total variation allowed for the fraction between the No. 8 and No. 16 sieves (table 1) is from 6 to 9 percent or only 3 percentage points. Another type of drastic slope limitation that generally reduces the flexibility of a specification is the requirement of a disproportionately large fraction between any two sieves. Such devices generally fail to serve any useful purpose and make specifications appear to be quite open and flexible when actually they are very difficult and expensive to meet.

It is evident that percentage passing specifications and percentage passing and retained specifications may have identical grading-band limits. Resulting mixtures obtained by either will generally be approximately the same, although many gradations meeting the former will not meet the latter. Regardless of the differences to be considered in analyzing the two systems, the method of plotting described here gives a reasonable comparison of requirements under the two systems. It also permits comparison of the gradations with ideal gradation curves.

The percentage passing specification is perhaps more easily understood and is more readily compared with the ideal gradation curves. It is also more convenient in predetermining graphically the gradation resulting from the combination of two separate aggregates (fine and coarse) and in checking the combined gradation against the controlling limits of the specifications.[1]

Difficulty is sometimes encountered in comparing bituminous concrete specifications because of the fact that some specifications give aggregate gradation requirements with separate indication of the amount of asphalt to be added, while other specifications include the asphalt with the aggregate in the gradation requirements. The latter method makes it difficult to predetermine that a particular aggregate, when mixed with the asphalt, will produce a mix complying with the specification.

The result desired in all cases is a bituminous concrete that will adequately serve under the anticipated traffic loads. Variations to fit the characteristics of local aggregates, the volume and weight of traffic to be accommodated, economic factors, and other influences undoubtedly are responsible for the wide range of controlling limits observed in different specifications. However, it would appear logical to prepare specifications under a uniform system in order that the experience of different engineers might be interchanged and a common meeting ground for discussion could be developed.

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[1] *Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft.* Public Roads Administration, 1943, pp. 184-186.

HIGHWAY STATISTICS, 1951

Available about April 4 will be the Bureau's *Highway Statistics, 1951*, the seventh of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of highways, and highway mileage. *Highway Statistics, 1951*, will be sold by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 60 cents a copy. The full series of the annual bulletin are available from the Superintendent of Documents, as indicated on the inside back cover of Public Roads.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 28, 1953

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$12,740	$24,069	$12,366	432.4	$7,145	$3,618	231.6	$30,693	$15,606	382.5	$61,907	$31,590	1,046.5
Arizona	2,048	8,514	6,080	176.6	1,073	718	16.9	6,027	3,465	71.3	15,614	10,263	264.8
Arkansas	10,419	10,415	5,528	304.6	1,590	817	77.5	12,479	6,405	217.7	24,484	12,750	599.8
California	5,886	44,275	22,094	191.4	13,263	6,806	52.4	89,948	43,339	201.8	147,486	72,239	445.6
Colorado	9,257	4,958	2,815	91.9	2,320	1,161	46.1	10,073	5,049	124.7	17,351	9,025	262.7
Connecticut	8,190	1,320	710	4.7	3,904	1,928	17.8	10,242	5,235	16.6	15,466	7,873	39.1
Delaware	3,509	650	325	.8	40	16		7,610	3,846	35.2	8,300	4,187	36.0
Florida	10,248	13,365	6,710	177.9	8,205	4,378	72.9	14,469	7,323	280.2	36,039	18,411	531.0
Georgia	15,223	11,325	5,884	255.1	6,212	3,103	42.1	34,945	16,727	568.6	52,482	25,714	865.8
Idaho	7,130	10,200	6,141	265.7	1,737	1,100	46.7	7,567	4,853	107.6	19,504	12,094	420.0
Illinois	19,420	46,149	24,759	345.2	25,549	13,125	226.3	50,186	26,082	364.1	121,884	63,966	935.6
Indiana	20,032	29,815	15,338	131.9	8,580	4,242	128.9	23,719	12,885	136.3	62,114	32,465	397.1
Iowa	9,337	19,449	10,134	456.5	3,955	1,991	131.4	9,279	4,688	403.6	32,683	16,813	991.5
Kansas	10,611	12,293	6,068	1,033.0	7,928	3,831	377.3	10,798	5,404	630.7	31,019	15,303	2,041.0
Kentucky	11,359	12,132	6,570	163.3	5,252	2,924	103.7	13,562	6,764	232.7	30,946	16,258	499.7
Louisiana	7,256	15,764	7,880	120.6	6,219	3,026	37.5	24,719	11,905	125.1	46,702	22,811	283.2
Maine	4,510	6,231	3,361	24.8	504	370	.8	10,824	4,998	80.7	17,559	8,729	106.3
Maryland	10,732	7,226	3,300	53.4	1,875	771	14.8	9,783	5,267	40.6	18,884	9,338	108.8
Massachusetts	11,609	3,602	1,946	12.2	4,403	2,194	8.6	40,159	19,051	38.0	48,164	23,191	58.8
Michigan	18,761	17,954	9,190	331.4	6,206	3,037	73.0	57,463	24,841	250.7	81,623	37,068	655.1
Minnesota	10,411	17,313	9,227	1,473.5	4,039	2,035	113.8	8,977	4,959	191.5	30,329	16,221	1,778.8
Mississippi	9,684	11,670	5,956	464.1	5,289	2,650	121.7	15,513	8,261	411.9	32,472	16,867	997.7
Missouri	13,407	31,048	15,931	999.3	16,104	8,032	194.5	31,632	16,807	351.7	78,784	40,770	1,545.5
Montana	14,681	8,082	4,875	232.4	1,170	705	20.0	14,048	8,432	229.8	23,300	14,012	482.2
Nebraska	18,620	9,470	5,115	466.9	3,807	2,455	55.0	9,842	4,851	247.3	23,119	12,421	769.2
Nevada	6,797	5,226	4,376	148.8	1,094	914	17.4	3,966	2,983	120.7	10,286	8,273	286.9
New Hampshire	3,553	3,168	1,584	20.6	847	390	3.8	5,111	2,682	31.3	9,126	4,656	55.7
New Jersey	7,412	9,314	4,511	39.6	14,621	6,785	2.5	27,868	13,790	34.3	51,803	25,086	76.4
New Mexico	3,247	7,706	4,927	202.0	2,995	1,899	68.2	6,301	4,031	184.4	17,002	10,857	454.6
New York	41,382	78,302	41,418	115.9	39,711	18,735	66.8	95,904	44,339	300.7	213,917	104,492	483.4
North Carolina	14,852	16,398	7,772	229.0	5,283	2,552	91.3	26,477	12,713	462.9	48,158	23,037	783.2
North Dakota	4,068	12,311	6,354	1,014.5	2,050	1,025	203.6	5,185	2,628	445.2	19,546	10,007	1,663.3
Ohio	24,470	16,964	8,835	101.5	9,813	5,207	28.6	78,717	39,326	107.7	105,494	53,368	237.8
Oklahoma	14,536	7,569	4,093	107.4	4,781	2,722	80.0	19,679	10,160	278.6	32,029	16,975	466.0
Oregon	5,358	1,942	1,101	24.9	3,996	2,410	58.2	11,807	6,683	161.2	17,745	10,194	244.3
Pennsylvania	20,903	38,382	19,184	78.5	11,103	5,544	44.1	79,960	39,765	164.7	129,445	64,493	287.3
Rhode Island	2,312	5,061	2,530	40.6	1,106	553	8.6	17,996	9,377	19.7	24,163	12,460	68.9
South Carolina	8,128	10,252	5,496	206.3	2,111	1,083	75.4	15,727	8,085	363.1	28,090	14,664	644.8
South Dakota	1,637	13,308	7,810	896.1	3,740	2,113	177.8	6,234	3,895	346.1	23,282	13,818	1,420.0
Tennessee	14,142	8,108	4,027	320.2	5,606	2,809	76.9	30,006	13,771	295.8	43,720	20,607	692.9
Texas	29,672	4,793	2,271	131.4	15,454	8,274	283.9	54,998	29,803	1,103.5	75,245	40,348	1,518.8
Utah	3,933	4,209	3,239	51.6	2,889	2,175	75.1	7,395	5,629	101.8	14,493	11,043	228.5
Vermont	2,718	3,318	1,843	30.4	794	397	6.4	6,695	3,340	44.6	10,807	5,580	81.4
Virginia	8,481	14,433	7,181	120.6	6,270	3,110	137.7	26,788	12,790	206.3	47,491	23,081	464.6
Washington	2,842	22,046	11,583	213.5	440	245	19.0	9,407	5,271	70.8	31,893	17,099	303.3
West Virginia	6,093	5,970	3,018	31.7	6,304	3,210	30.1	16,504	8,250	133.6	28,778	14,478	195.4
Wisconsin	9,470	21,743	11,401	239.9	3,348	1,662	51.3	30,053	15,403	296.0	55,144	28,466	587.2
Wyoming	2,824	4,379	2,873	75.7	1,188	741	20.0	5,782	3,869	108.8	11,349	7,483	204.5
Hawaii	2,588	2,696	1,325	7.9	582	291	4.9	11,788	5,264	34.7	15,066	6,880	47.5
District of Columbia	4,919	10,210	4,489	.9	1,964	608	.3	4,646	2,320	.5	16,820	7,417	1.7
Puerto Rico	5,891	9,499	4,640	67.1				12,878	6,127	46.8	22,377	10,767	113.9
TOTAL	527,308	714,596	376,184	12,726.2	294,459	150,487	3,843.2	1,172,429	589,337	11,204.7	2,181,484	1,116,008	27,774.1

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Public Roads

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Braking test from high speed at Andrews Air Force Base, Maryland

Public Roads

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
FRANCIS V. DU PONT, Commissioner

Braking Distances of Vehicles from High Speed and Tests of Friction Coefficients

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
O. K. NORMANN
Chief, Traffic Operations Section

The limited tests to examine braking distances of vehicles from high speed and to measure coefficients of friction, reported in this article, were of sufficient scope to throw doubt on some of the beliefs heretofore commonly accepted.

What braking force or deceleration rate should be expected? Are stopping distances from high speeds longer than generally accepted as correct? What causes the wide variation in braking action of different vehicles, and of the same vehicle in different trials? Are deceleration rates attainable by vehicles at high speeds uncomfortable to the passengers, so long as the vehicle follows a straight path? To what extent do brake fade and other factors affect stopping distances?

Tentative indications as to the answers to these questions were found in the braking tests described here, but they serve principally to point the need for far more extensive studies that should involve the cooperation of the automotive and tire industries.

The results of the friction coefficient measurements that were made are useful as a pilot study to illustrate the required magnitude of any investigation of the interrelation of stopping distances and friction coefficients. It appears necessary to consider variation in the nonskid qualities of both tires and road surfaces to obtain the most effective improvement in operating safety.

THE late Ernest E. Wilson, at that time director of the General Motors proving ground, reported in December 1940 the results of tests to determine deceleration distances for high-speed vehicles. He used 15 passenger cars in perfect condition and 8 highly experienced test drivers to obtain braking distances for speeds ranging from 50 to 70 miles per hour. Since 1940, about 35 million passenger cars have been built in the United States, but few if any results of tests to determine braking distances from high speeds have been reported. There have been numerous reports for tests made from speeds of 20 to 40 miles per hour, but passenger cars are now being operated at an average speed of about 52 miles per hour on our main rural highways, with about 12 percent exceeding 60 miles per hour, and an occasional vehicle traveling in excess of 80 miles per hour.

One possible reason that more high-speed tests have not been conducted is the common assumption that any passenger car with brakes in good condition can lock all four wheels and that shorter stopping distances can be realized only through improving the texture of road surfaces to obtain higher coefficients of friction, especially for wet surfaces, and by avoiding the use of tires that have worn smooth. Although a more critical skidding condition usually exists when a surface is wet than when dry, about three out of four fatal accidents and two out of three of all accidents occur on dry surfaces. Many of these no doubt could have been avoided if the drivers had been able to stop a little sooner.

To obtain current information on braking distances from high speeds, the Bureau of Public Roads in 1949 conducted a series of tests on 53 vehicles representing 10 of the most common makes. Twelve of the cars were Government vehicles owned by the Bureau of Public Roads. The other 41 were private cars owned by employees of the Bureau of Public Roads who volunteered to participate in the tests. Each vehicle was driven during the test by the person who normally operated the vehicle.

Stops similar to those that would have been made in an emergency were made by all of the drivers from speeds of 20 and 40 miles per hour. Most drivers also made emergency stops from 60 miles per hour, and the drivers of the Government vehicles and some of the private cars made stops from the highest attainable speed, which was generally over 70 miles per hour. Data were obtained for a total of 214 stops, including 14 at speeds exceeding 75 miles per hour and 7 at 90 miles per hour.

Test Procedure

The tests were conducted on a concrete taxiway 6,500 feet long and 50 feet wide at Andrews Air Force Base. The surface had a broomed finish and was free of oil drippings since it had been used very little. About 2,000 feet from the end of the taxiway, a rubber tube with an air-switch on one end was stretched across the surface. As the front wheels of a vehicle passed over this tube, a solenoid was actuated which turned on a brilliant light 700 feet away on the right-hand side of the surface. This light acted as a signal to inform the driver to apply his brakes as quickly as possible. Immediately ahead of the tube which actuated the light were three other tubes connected to a recording speedmeter located in a panel truck parked at one side of the surface. Accurate speeds could thus be obtained immediately prior to the brake application for vehicles traveling within the range of 15 to 100 miles per hour.

Each vehicle to be tested was equipped with a detonator mounted on the front bumper (fig. 1) and actuated electrically through a switch on the brake pedal (fig. 2). When the brake pedal was touched, the detonator fired a .22-caliber cartridge, ex-

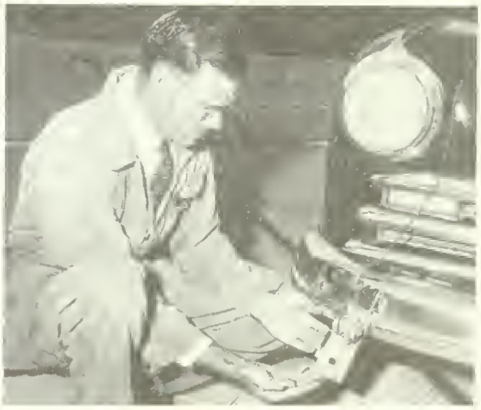

Figure 1.—Mounting detonator on vehicle.

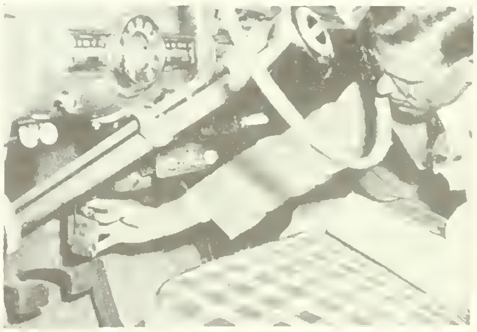

Figure 2.—Installing switch on brake pedal.

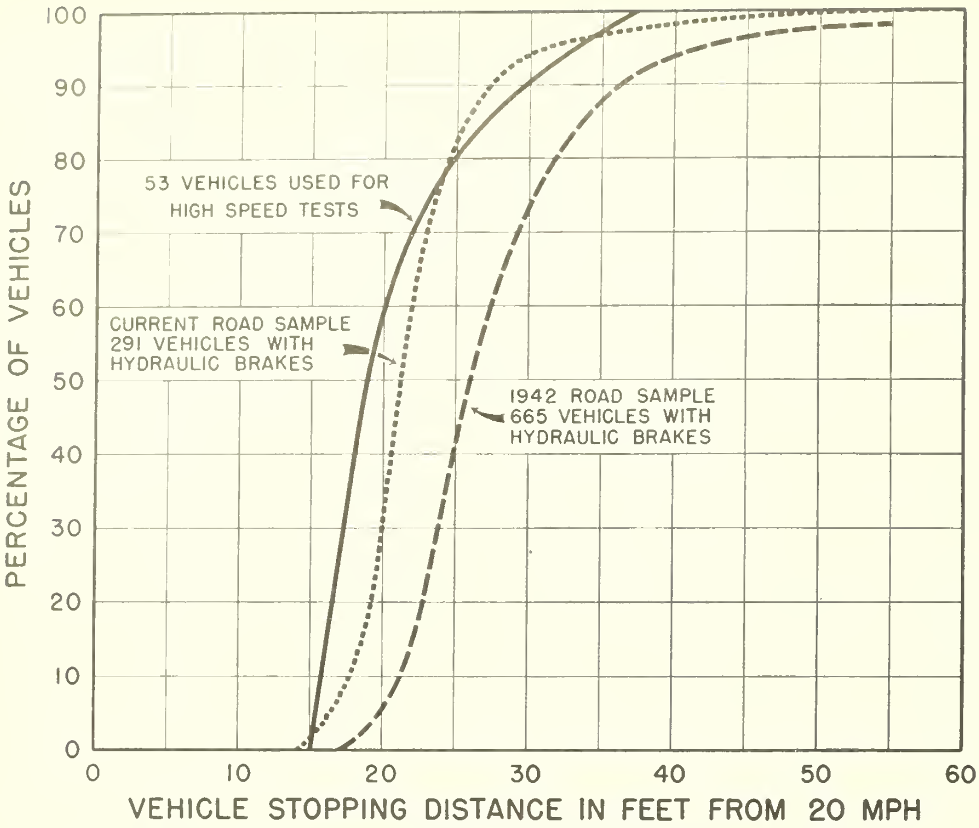

Figure 3.—Stopping distances for various groups of vehicles sampled.

pelling a pigment-filled capsule which left a yellow mark 5 inches in diameter on the pavement to indicate the point at which the brakes were first applied. The distance between the tube which actuated the light and the yellow mark was the distance traveled during the driver's reaction time, and the distance between the yellow mark and the point where the vehicle came to a stop was the braking distance. Measurements were made by use of a scale marked directly on the concrete surface.

Prior to testing each vehicle, the following information was recorded: Year model and make of vehicle, present mileage on vehicle, mileage when brakes were last relined or adjusted, distance between floor boards and brake pedal when depressed, condition of tires, and name and age of driver.

During each brake test, observers stationed along the taxiway noted whether any wheels skidded and, if so, which wheels locked. The drivers were requested to be in high gear before reaching a speed of 20 miles per hour, to accelerate as fast as possible to the predetermined braking speed, to continue at that speed until they saw the spotlight illuminated, and then to come to a stop as quickly as possible. No instructions were given the drivers regarding use of the clutch pedal and no observations were made as to when the clutch pedal was depressed during the stops.

An observer riding in the vehicle noted the time required to accelerate to various speeds and the speedometer reading immediately before the brakes were applied. The recorded speedometer readings were later adjusted to conform with actual speeds. The observer also kept the driver informed of the speedometer reading so that the driver could keep his eyes trained on the road and spotlight. After the test, the observer questioned the driver as to whether he considered the stop a comfortable one, whether he thought the stop could have been made in a shorter distance, and whether he applied his maximum force to the brake pedal during the stop. The observer also measured the distance between the floor board and the depressed brake pedal.

Performance Better Than Average

Only 12 of the 53 vehicles used for the high-speed brake tests were new vehicles; 3 were 10 years old. The average age of the 53 test vehicles was 3.4 years, less than half the average age of 7 years for all passenger cars registered in the United States during 1951. All of the drivers participating in the test considered their brakes in good condition, and it will be shown that the vehicles participating in the brake tests could be stopped from a speed of 20 miles per hour in shorter distances than the average vehicle being operated on public highways.

Comparisons of stopping distances from a speed of 20 miles per hour for the 53 vehicles used in the high-speed tests with corresponding values for vehicles included in other comprehensive studies are shown in figure 3. The results for the 1942 road sample and the current road sample were obtained by conducting brake tests on vehicles selected at random from everyday traffic on main rural highways. Each vehicle selected at random was subjected to three or more emergency stops from 20 miles per hour on dry concrete pavements.

It may be noted from figure 3 that there has been a substantial improvement since 1942 in the brake performance of vehicles in operation on our highways. In 1942, only 40 percent could be stopped in less than 25 feet and 13 percent required more than 35 feet. Today, 83 percent can be stopped in 25 feet and only 3 percent require more than 35 feet. There are still, however, some vehicles being operated on our highways with brakes in such poor condition that they cannot be stopped from 20 miles per hour in 60 feet, more than four times the distance required for some vehicles.

None of the vehicles used for the high-speed tests required more than 37 feet to stop from 20 miles per hour. On an average, they showed somewhat better brake performance from 20 miles per hour than those selected for the current road sample. This was to be expected, since most of the drivers who volunteered for the high-speed tests thought, no doubt, that they and their vehicles would perform as well or better than average. Most of the drivers undoubtedly volunteered because they were willing to spend some time and money in helping to obtain useful facts relating to

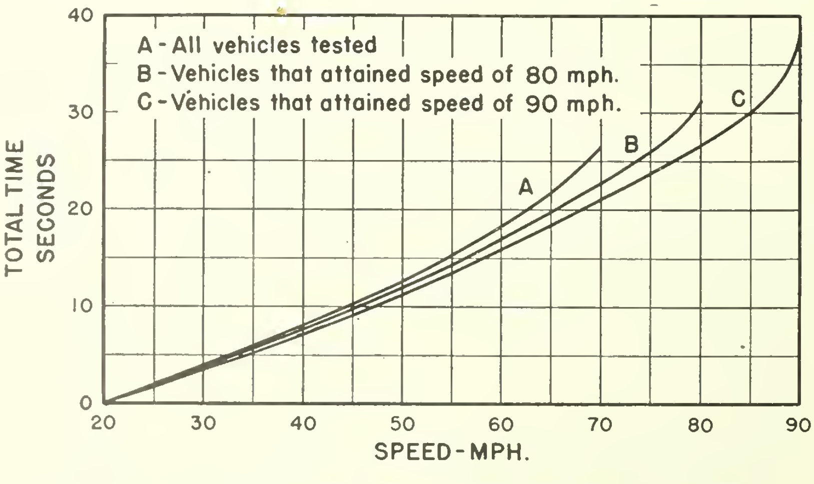

Figure 4.—Time to accelerate from 20 miles per hour in high gear.

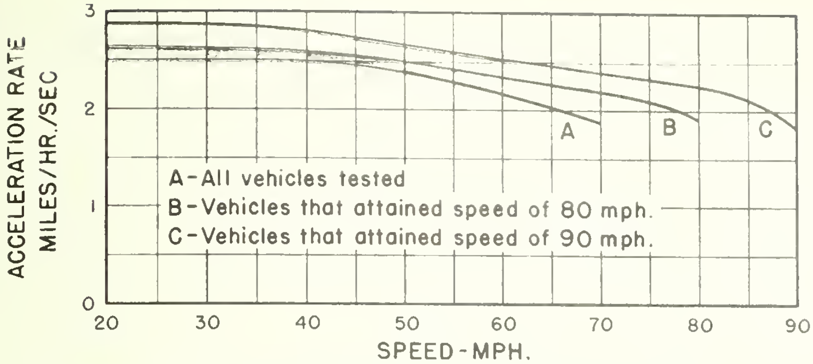

Figure 5.—Average acceleration rate from 20 miles per hour.

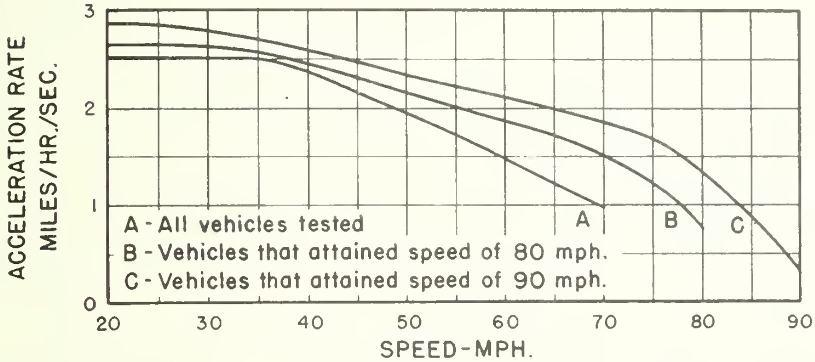

Figure 6.—Instantaneous acceleration rates at various speeds.

one of the most necessary elements of highway safety, the ability of vehicles to stop from high speeds. It is doubtful, however, that any one of the drivers would have participated in the test had he known that his performance would have been the worst of the lot.

Acceleration Rates

Before discussing the more involved results of the brake-performance tests, it seems desirable to dispose of a few of the results of these tests that may be termed byproducts. These relate to the acceleration rates of passenger cars, reaction times of drivers, and accuracy of speedometers. Figures 4, 5, and 6 relate to the acceleration rates of passenger cars. In these figures, the passenger cars have been classified into three groups. Group A includes all of the vehicles tested; group B includes only the vehicles that reached a top speed of about 80 miles per hour on the test course; and group C includes only the vehicles that reached a top speed of 90 miles per hour on the test course.

Figure 4 shows the total time required to accelerate to any given speed from a speed of 20 miles per hour, in high gear. It is also possible to determine, by subtraction, the time to accelerate from a given speed to any higher speed within the range of the chart. The vehicles capable of a 90-mile-per-hour speed (group C) could accelerate from 20 to nearly 70 miles per hour in approximately the same time as they required to accelerate from 70 to 90 miles per hour. They traveled an average of 1,450 feet in going from 20 to 70 miles per hour, and 1,850 feet in going from 70 to 90 miles per hour.

The average acceleration rates while increasing from a speed of 20 miles per hour to a higher speed are shown in figure 5. The average rate of acceleration decreases as the speed increases, although the difference is slight between 20 and 40 miles per hour.

The instantaneous acceleration rates at various speeds are shown in figure 6. The average vehicle (group A) had an acceleration rate of 2.5 miles per hour per second at speeds between 20 and 35 miles per hour, whereas the vehicles that could reach 90 miles per hour (group C) had a maximum instantaneous acceleration rate of 2.8 miles per hour per second at a speed of 20 miles per hour. Rates of 2.5 and 2.8 miles per hour per second are equivalent to 3.7 and 4.1 feet per second per second, respectively. As the speed approaches the top speed of a car, the acceleration rate approaches zero. From the curves of figure 6 it appears that the cars which reached a speed of 90 miles per hour could have eventually reached about 92 miles per hour, and those that reached a speed of 80 miles per hour could eventually have increased their speeds to 85 miles per hour.

Information regarding the acceleration rates of passenger cars is used in connection with many highway design problems relating to traffic operations. In recent years the principal source of such information has been the results obtained for six passenger cars tested in 1938. The results for the 53 cars in current use indicate that acceleration rates at the present time are 20 to 30 percent higher than for the six cars tested in 1938.

Driver Reaction Time

The measurements of driver reaction time and distance that were made during the braking-distance tests from high speeds may be considered as absolute minimums. Each driver was aware of the approximate time that the stop was to be made and was poised for the occasion. Only the reaction time for the initial test run of each driver has been used for this analysis. During subsequent tests, several of the drivers anticipated the time that the signal light would be illuminated and had removed their foot from the accelerator prior to reaching the road tube that actuated the light.

Figure 7 shows the distribution of reaction times for the 53 drivers. Some drivers

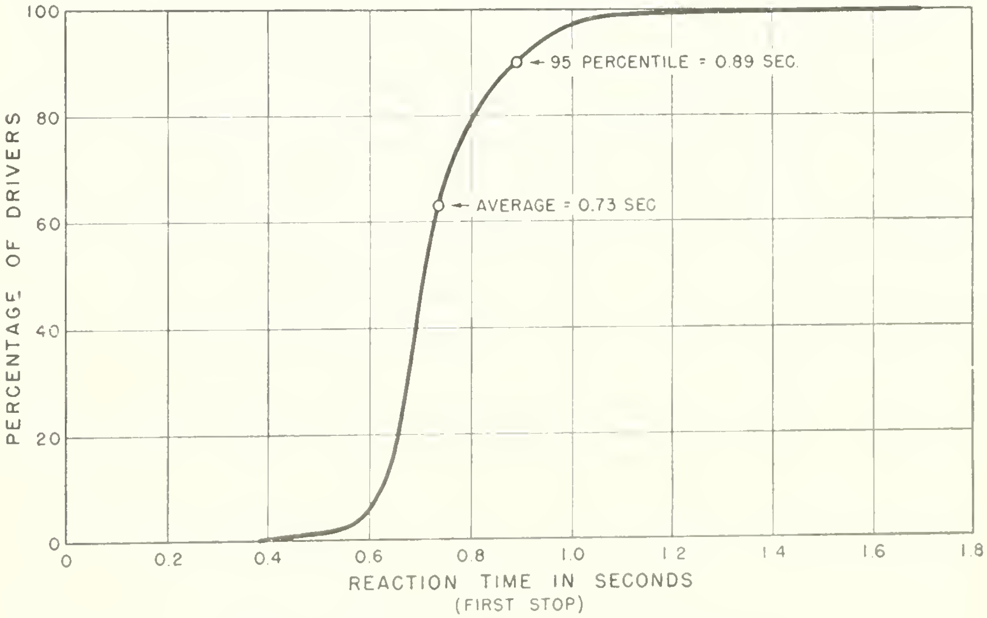

Figure 7.—Reaction times during high-speed tests.

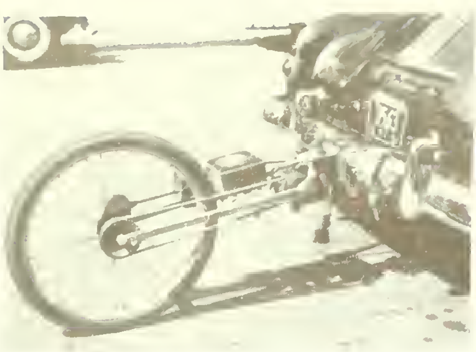

Figure 8.—Fifth wheel used to obtain time-distance curves.

reacted and moved their right foot from the accelerator to the brake pedal in 0.4 second. One driver required 1.7 seconds. Repeated tests on the drivers who had the longer reaction times gave consistent results which eliminated the possibility that these drivers misinterpreted the instructions. All except the one driver had a reaction time of less than 1 second for the conditions of these tests. The average reaction time was 0.73 second and 95 percent of the drivers reacted in less than nine-tenths of a second, which is consistent with the results of other studies.

Most Speedometers Inaccurate

It was interesting to find that more of the speedometers were low than high when compared with the actual speeds of the vehicles. This is contrary to the common belief that speedometers have a tendency to indicate higher speeds than the actual speeds. If it is considered that a speedometer is correct when it registers within 2 miles per hour of the actual speed, which is as close as the readings on many of the present models can be determined, the following were the results for the 53 vehicles:

19	speedometers correct at all speeds tested.
6	speedometers more than 2 miles per hour high.
28	speedometers more than 2 miles per hour low.

The average error for the 6 speedometers that were high was 6.7 percent for speeds below 50 miles per hour and 6.1 percent for speeds above 50 miles per hour. At no speed did any of these speedometers have an error of more than 10 percent.

The average error for the 28 speedometers that were low was 12.1 percent for speeds under 50 miles per hour and 10.1 percent for speeds over 50. Four of the speedometers were between 20 and 24 percent low at all speeds: These indicated 57 to 60 miles per hour when the vehicles were actually traveling 75 miles per hour. The methods used for conducting the test eliminated the possibility that these errors resulted from a lag in the speedometers.

Table 1.—Results of braking-distance tests of a single vehicle.

Test No	Initial speed	Braking distance	Skid marks
	M.p.h.	*Feet*	
1	18	14	Four wheels
6	19	14	Do.
16	30	48	None
12	30	45	Four wheels
11	30	46	Light
17	30	50	Do.
2	45	80	Four wheels
3	45	133	None
7	53	137	Four wheels [1]
8	53	162	None
13	79	540	None
14	78	468	None
15	80	564	None
18	80	400	Light [2]
19	81	400	Four wheels [3]
4	90	608	None
5	90	569	None
10	90	482	Four wheels [1]
9	90	640	None [4]

[1] Vehicle made a sharp dive.
[2] Very smooth stop.
[3] Four wheels locked for 251 feet.
[4] Attempt made to fan brakes.

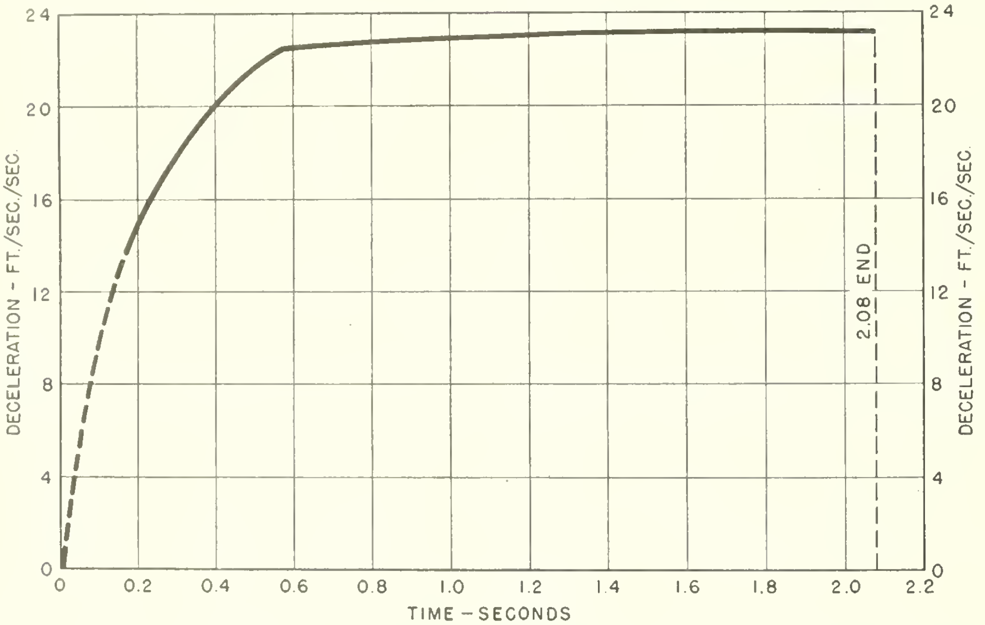

Figure 9.—Deceleration rates during stop from 30 miles per hour (average of two tests).

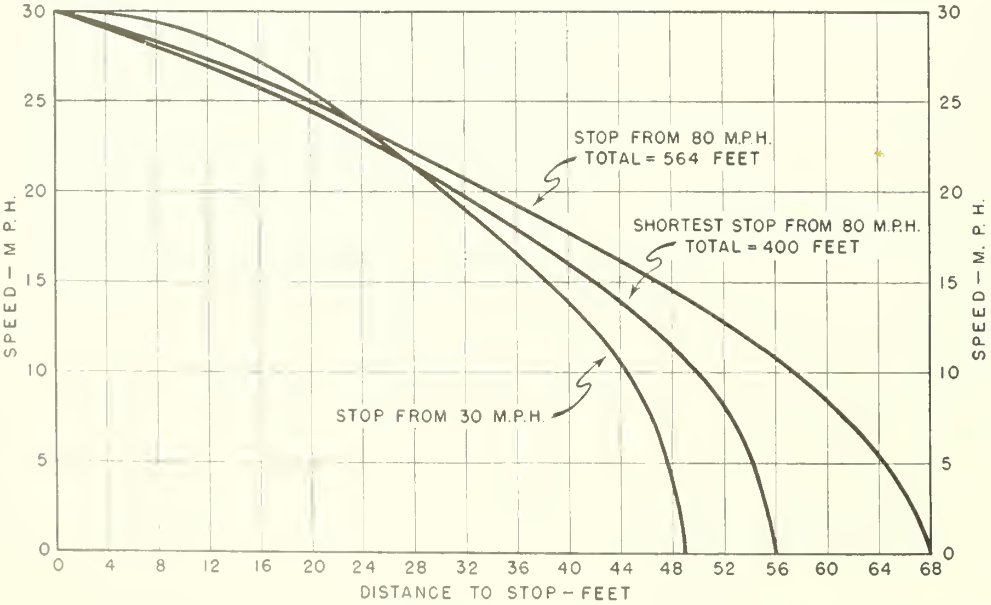

Figure 10.—Braking distance from 30 miles per hour with initial speeds of 30 and 80 miles per hour.

Brake Fade

In all of the brake tests, actual measurement was made of the distance from the chalk mark on the pavement, indicating the point where the operator touched the brake pedal, to the point where the car came to rest. In addition, in a few of the tests a fifth wheel and chronograph were mounted on the test vehicle to obtain a time-distance record during each stop (fig. 8). From the time-distance record, deceleration rates were determined. Figure 9 shows a curve representing the average rates of deceleration during two stops from 30 miles per hour by one vehicle. In the one case, the stopping distance was 48 feet, and in the other it was 50 feet.

Figure 9 shows that the deceleration rate reached 22 feet per second per second within 0.6 second after the operator touched the brake pedal. It is not known whether there was any deceleration within the first 0.1 second, but after 0.2 second had elapsed, the vehicle was decelerating at a rate of 15 feet per second per second. After 0.6 second there was only a very slight increase in the deceleration rate until the vehicle came to a stop.

During several of the stops from high speeds, the brakes seemed to fade shortly after being applied, making it appear as though the vehicle had very poor brakes. Brake fade may be defined as a temporary reduction in brake effectiveness resulting from heat. In such a case, the distance traveled to bring the vehicle to a stop after its speed had been reduced to 30 miles per hour seemed exceptionally long. In fact, one driver thought his brakes were not functioning during the latter part of a stop from 60 miles per hour. Fading usually did not appear to be pronounced during stops below 70 miles per hour except when several stops were made within a few

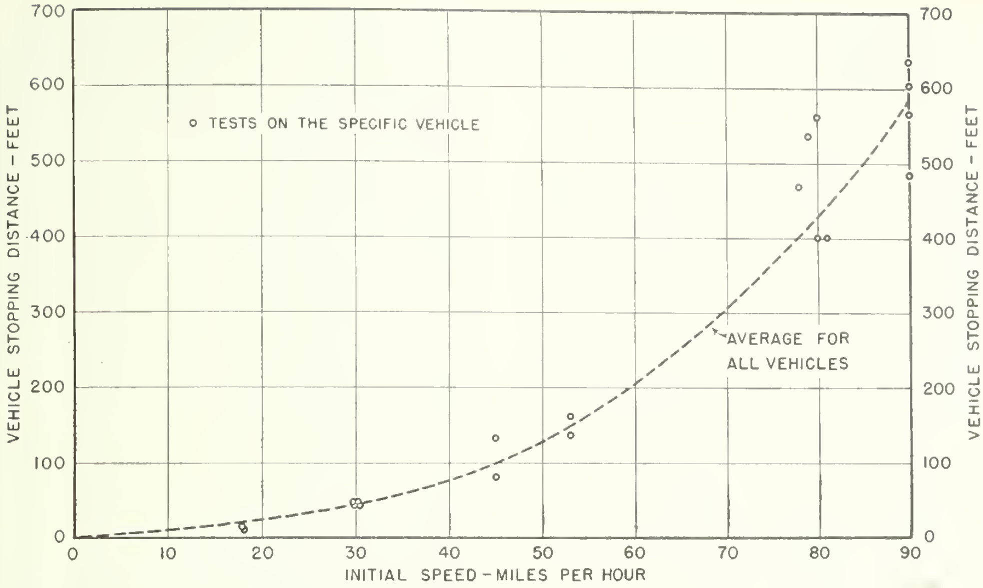

Figure 11.—Results of brake tests made on one specific vehicle as compared to all vehicles.

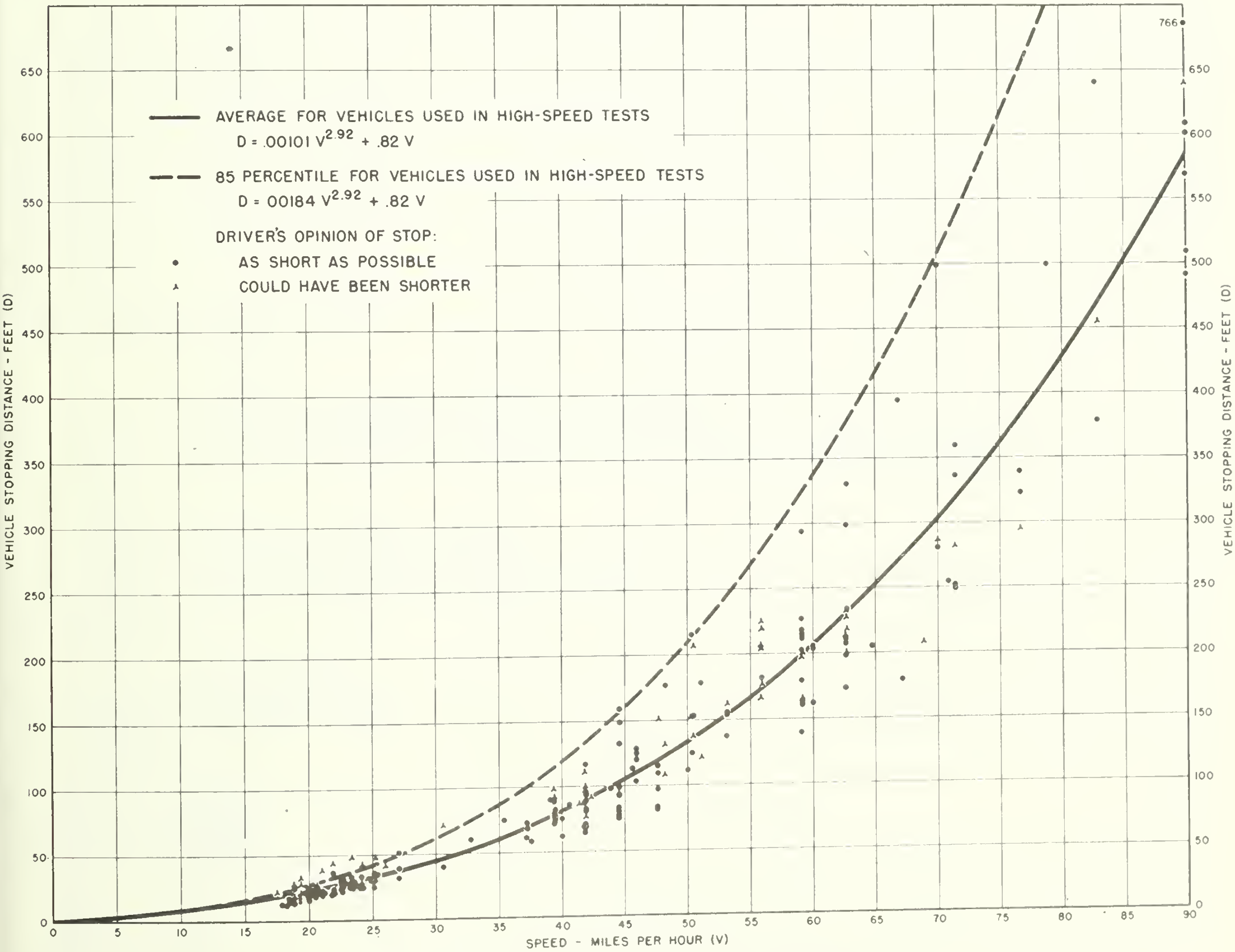

Figure 12.—Braking distances during high-speed tests.

minutes without giving the brake drums time to cool between the brake applications. The general procedure for these tests was to allow the brakes time to cool between successive applications.

The distances required to bring a particular vehicle to a stop from a speed of 30 miles per hour when the initial speed was 30 miles per hour in one case and 80 miles per hour in two other cases are shown in figure 10. The one stop from an initial speed of 80 miles per hour was made in 400 feet, the shortest distance in five tries, whereas the other stop from 80 miles per hour was made in 564 feet, the longest distance in five tries. None of the wheels locked during any of the three stops illustrated in figure 10, although the operator applied the maximum force possible to the brake pedal in each instance.

The speed-distance curve in figure 10 for the stop from an initial speed of 30 miles per hour is the average data for the same stops as those illustrated in figure 9. The curve in figure 10 shows that at the beginning of these stops there was a decrease in speed of only 1 mile per hour during the first 8 feet of travel. Shortly thereafter, the vehicle's speed decreased about 1 mile per hour for each 1.6 feet of travel, and immediately before coming to a stop the speed decreased 4 miles per hour while the vehicle traveled about 1 foot.

With an initial speed of 80 miles per hour, the vehicle was decelerating a little less than 1 mile per hour for each 4 feet of travel while going 30 miles per hour. During the remainder of the two stops, the speed decreased at a much lower rate for the same travel distance than during the stop made from an initial speed of 30 miles per hour.

One of the stops made from an initial speed of 80 miles per hour required 68 feet to bring the vehicle to a stop after the speed was reduced to 30 miles per hour, while in the other case the corresponding distance was 56 feet. The difference between these two distances and the 49 feet required for an initial speed of 30 miles per hour may be attributed to brake fade. In the one case the difference was 19 feet or 39 percent of the normal stopping distance for this vehicle from 30 miles per hour.

The tests on which the information in figure 10 was based were by no means the worst examples of brake fade that occurred during the series of tests. They were the only high-speed tests for which this type of data were obtained and have been presented to illustrate one reason that stopping distances do not vary exactly as the square of the speed.

Variation in Brake Performance

Under identical conditions, brake performance at high speeds was not always the same. The variation in braking distances for one vehicle with the same driver on the same surface is illustrated by the data shown in table 1. Figure 11 shows the stopping distances from various speeds for this one vehicle as small circles and the average stopping distances for all 53 vehicles tested as a dashed curve.

It may be seen from figure 11 that the results for this one vehicle, which was subjected to more tests than any other vehicle, compare closely to the average values. Fading of the brakes was especially pronounced during the three stops above the average line for speeds in the neighborhood of 80 miles per hour and during the two stops above the line at 90 miles per hour. These stops required considerably longer distances than other stops made by the same vehicle at corresponding speeds.

Table 1 also reports a record of the skid marks made by the test vehicle. It must be remembered that during all of these tests, except Nos. 9 and 19, the driver applied the maximum possible pressure to the brake pedal over the entire stopping distance. In some instances the wheels locked, leaving skid marks from all four wheels. In other instances none of the wheels locked. During some of the stops when the wheels did not lock, they were evidently not turning as fast as the car was traveling because light skid marks or tire traces were plainly visible, generally outlining the edges of the tire tread.

During the nine tests from speeds above 75 miles per hour, it was possible to lock the wheels only twice (Nos. 10 and 19). During test No. 9, an attempt was made to eliminate brake fade by fanning the brakes (removing all pressure from the brake pedal occasionally), but this apparently was not effective in reducing the total braking distance.

The results for tests Nos. 18 and 19 are especially significant. For both of these stops, which were made from approximately the same speed, the braking distance was 400 feet. Test No. 18 was a very comfortable stop, whereas the stop made during test No. 19 was the most dangerous one of the entire series.

During test No. 19, all four wheels locked at a point 31 feet beyond the point where the brakes were applied. The vehicle skidded for 151 feet, struck a construction joint in the pavement, and dived to the adjacent lane, at which time the brakes were released sufficiently to allow the wheels to turn. The vehicle then traveled 118 feet with the wheels turning and finally skidded 100 feet to a stop. The tires were so badly worn on one side as a result of this one stop that all four had to be replaced to eliminate a pronounced bumping as the vehicle was driven over a smooth surface.

The braking characteristics experienced during the 19 tests on this one vehicle indicate that if the wheels do not lock immediately after full pressure is applied, they cannot be locked at all. There evidently is enough heat developed with full brake pressure and the wheels turning so that fading soon takes place, especially at the higher speeds, and much longer stopping distances result than when the wheels lock. Whether or not it will be possible to lock the wheels at high speeds is unpredictable on a dry concrete surface of the type where these tests were conducted. Stops from high speeds with the wheels locked are, of course, dangerous from a viewpoint of maintaining control of the path of the vehicle. In an emergency, however, bringing the vehicle to a stop in the shortest distance possible may or may not be more important than the exact path of the vehicle, depending upon the circumstances. Even with the most careful and experienced drivers, such situations do arise.

The braking distance and initial speed recorded for each test conducted on all the vehicles are shown by the points in figure 12. In an attempt to determine the reason for the comparatively wide scatter of the points, the tests for which the drivers stated that they made the stop as fast as possible have been shown as dots, whereas the small triangular points represent tests for which the driver stated that he thought the stop could have been made in a shorter distance.

It should be noted that figure 12 shows a fairly even distribution of both types of points on both sides of the average curve. In fact, when the stopping distances that the drivers thought were not as short as possible were compared with those they thought were the shortest possible, it becomes evident that the driver's opinion is of little value. A summary of the results is shown in table 2. For speeds of 50 miles per hour or less, the braking distances for the stops made as fast as possible are slightly shorter than for the stops not made as fast as possible (table 2). For speeds above 50 miles per hour, the reverse is true, and the difference is more pronounced.

Comfort of Stop

It is also interesting to compare the actual braking distances with the drivers' opinions as to whether or not the stops could be considered comfortable. Invariably the more comfortable stops were made in the shorter distances even when stops by the same driver-vehicle combination were compared. Within the range of these studies it is therefore evident that other factors have a greater influence than the average deceleration rate on the comfort character-

Table 2.—Stopping distance related to driver opinion of performance

Initial speed	Stopping distance, when, in the driver's opinion, the stop—	
	Was made as fast as possible	Could have been made in a shorter distance
M.p.h.	*Feet*	*Feet*
20	25	34
40	86	90
50	126	146
60	210	206
70	311	259
80	464	357

istics of the stop. Based on the opinion of the observer who rode with the driver on each test, the most uncomfortable stops, or those most likely to throw an occupant against the windshield or dashboard, were the stops made from the lowest speed of 20 miles per hour. At high speeds, the brakes are evidently not capable of overcoming the angular momentum of the wheels and other revolving parts of the vehicle at a fast enough rate to cause a sudden and great change in the speed of a vehicle.

Using data for the tests in which the drivers applied the maximum brake pressures they could develop, it was found that no wheels locked in 42 percent of the tests, all wheels locked in 30 percent, and one or two wheels—generally the rear wheels—locked in 28 percent of the tests. Wheels were more apt to lock at the lower speeds than at the higher speeds. At speeds of 20 miles per hour, all four wheels locked in 35 percent of the tests, whereas at speeds exceeding 60 miles per hour the corresponding figure was only 19 percent. The common assumption that any passenger car with brakes in good condition is capable of locking all four wheels may be questioned. It may hold true for certain drivers on all types of surfaces or for all drivers on certain surfaces, but not for all drivers on all surfaces. The concrete surface on the taxiway had a higher coefficient of friction than the surface of any highway on which tests were made (as described later) in connection with this investigation. A dangerous condition would exist, however, if manufacturers provided brakes that would grab or lock the wheels too easily.

Square-of-Speed Rule in Error

The braking distance does not vary as the square of the speed. For example, the average stop from 30 miles per hour was made in 40 feet. With the braking distance varying as the square of the speed, a stop from 90 miles per hour would require only 360 feet, whereas the average stop from 90 miles per hour actually required 580 feet. Not one of the stops from 90 miles per hour was made in 360 feet. The shortest distance was 490 feet.

There are several reasons why the braking distances do not vary as the square of the speed. The effects of brake fade have already been discussed. Likewise it has been shown that full deceleration does not start immediately when the brake pedal is touched. It takes some time to depress the pedal as far as it will go and some additional time for the brake fluid to expand the brake shoes through the wheel cylinders. Other factors also affect the brake distance. While it is true that the kinetic energy of the vehicle in the direction it is traveling varies as the square of the speed, the rate at which brakes can absorb this energy and the additional angular kinetic energy in the wheels and other rotating parts is apparently limited.

The relation between speed and braking distance as obtained by these tests is shown in figure 12, in which two curves are presented. The lower curve shows average stopping distances, and the upper curve shows the 85-percentile stopping distances. The equation for the lower curve is:

$$D = 0.00101V^{2.92} + 0.82V$$

where D is the average stopping distance in feet and V is the speed in miles per hour.

The constants in this equation were obtained by assuming that the equation should have the form $D = aV^b + cV$. They were determined so as to minimize the sum of the squared deviations of the plotted points from the curve. The second term on the right-hand side of the equation represents the distance traveled during actions taking a constant length of time, such as the time to depress the brake pedal and the time required for the brake cylinders to expand the brake shoes. This equation fits the data better than any other type of equation that was investigated and far better than one based on the assumption that braking distances vary as the square of the speed.

The 85-percentile curve in figure 12 shows the distance within which 85 stops out of 100 can be made and not the distance within which 85 percent of the vehicles can always stop. It has been based on observed data up to a speed of 40 miles per hour. Above 40 miles per hour there were so few tests that the 85-percentile curve has been based on sound statistical procedures assuming that the second term, $0.82V$, which consists largely of brake-application distance, and the exponent in the first term would be the same as for the average curve. It was also known that most of the vehicles with poor brake performance at the lower speeds were not tested at the higher speeds. In fact, only 3 of the 11 vehicles that required more than the 85-percentile distance to stop from a speed of 20 miles per hour participated in the tests at speeds above 40 miles per hour. Also, an analysis of the stopping

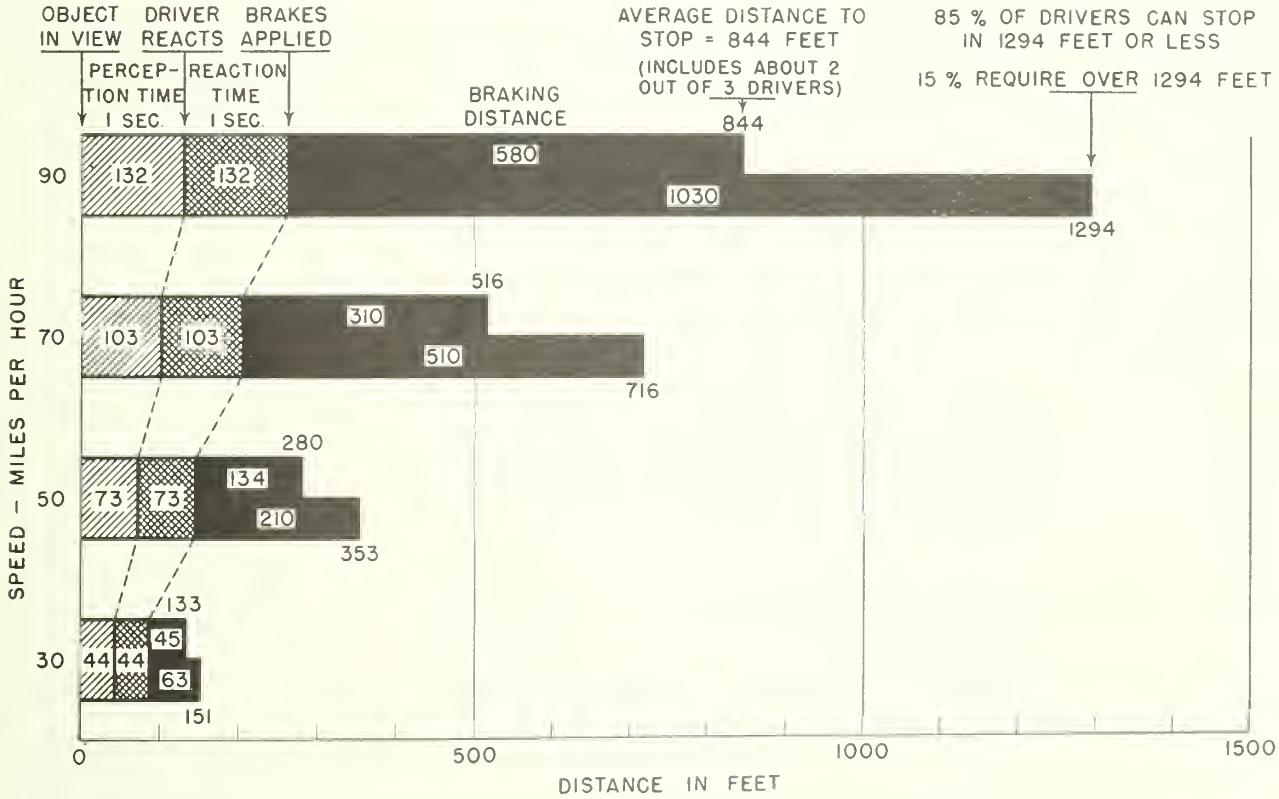

Figure 13.—Driver stopping distance on dry concrete.

distances of the vehicles that did participate at the high speeds shows a marked tendency for those having the longer stopping distances at the lower speeds also to have the longer stopping distances at higher speeds. The portion of the 85-percentile curve above 40 miles per hour therefore represents the best estimate that can be made on a basis of these tests even though few of the points shown in figure 12 represent stopping distances longer than the 85-percentile values.

Equations that are somewhat easier to solve and give approximately the same results as the equations shown in figure 12 are as follows, for the average and 85-percentile curves, respectively, with D and V being in the same units as for the exact equations:

$$D = 0.00069V^3 + V$$
$$D = 0.00126V^3 + V$$

An appreciation of the effect of speed on the distance required to stop a passenger car may be obtained from figure 13. The 1-second perception time and the 1 second for reaction time are certainly minimum values for drivers under actual operating conditions except possibly under congested traffic conditions when the time required by a driver to perceive the illumination of the stop light on a preceding vehicle might possibly be somewhat less than 1 second. Considering the conditions under which these data were obtained, the driver stopping distances shown in figure 13 can be considered as absolute minimums for use in determining standards of design for safe highways. If any safety factor is applied, longer driver stopping distances must be used. For certain elements of design the average values might be applicable, but safe conditions generally will not be attained unless driver stopping distances at least as high as the 85-percentile values are used.

Coefficients of Friction

Some surfaces when dry do not have as high a coefficient of friction as a concrete surface, and no road surface included in these tests had as high a coefficient of friction when wet as a dry concrete surface. The 85-percentile values for the driver stopping distance as shown in figure 13 are applicable to all road surfaces, however, where the coefficient of friction that can be developed between the tires and the road surface is equal to or greater than the lowest equivalent coefficient of friction utilized in these tests by the vehicles in making 85 percent of the stops.

The average coefficients of friction utilized by the vehicles over their entire braking distances on the dry concrete surface are shown in figure 14. These averages were calculated by use of the equation $d = V^2 \div 30f$ where d is the braking distance in feet, V is the initial speed in miles per hour, and f is the average coefficient of friction developed between the tires and the road surface over the entire braking distance. Since the braking distance includes the distance traveled during the brake application time, the average utilized coefficient of friction increases as the speed increases from 20 to 30 miles per hour because the brake application distance becomes an increasingly smaller portion of the total braking distance.

While making stops from 20 miles per hour, there was at least one vehicle that utilized an average coefficient of friction of 0.88 over its braking distance. The average coefficient of friction utilized by the average vehicle was 0.65, and the vehicles that required the 85-percentile braking distance utilized an average coefficient of 0.48. Likewise, the corresponding coefficients of friction that were utilized for the stops from 90 miles per hour were 0.57, 0.46, and 0.26 respectively. The maximum friction coefficient during any one stop is always greater than the average for the entire braking distance. For example, during the stop from 30 miles per hour as shown in figure 10, the vehicle could have stopped in the 49 feet if a coefficient of friction of 0.61 had been utilized over the entire distance, calculated on the same basis as the curves of figure 14. Actually, however, the maximum coefficient of friction developed during this same stop was 0.72 since the maximum deceleration rate was 23.2 feet per second per second as shown in figure 9. It is evident, therefore, that coefficients of friction greater than the average values shown in figure 14 were developed between the tires and the road surface during the braking-distance tests.

In an effort to determine the actual coefficient of friction of the taxiway surface and to compare this value with the coefficients utilized by the vehicles in braking, equipment was developed to measure friction coefficients of road surfaces. This equipment and other tests for which it was used will be described after the results of friction tests on the taxiway surface have been discussed.

Tires and Friction Factors

Coefficients of friction between the taxiway surface and one particular set of tires under dry conditions were found to be as shown in the column of table 3 headed "first set of tires." It was evident that if these were the correct coefficients of friction for the taxiway surface, it would have been impossible to make many of the stops within the distances recorded during the brake tests. Average coefficients of friction at least as high as those shown in figure 14 were necessary to stop the vehicles within the recorded distances, excluding the braking effect of air resistance.

The only explanation for the wide discrepancy seemed to be that the tires on the vehicle used to measure the friction coefficients were not as resistant to skidding as the tires on some of the vehicles used for the brake tests. This was confirmed by interchanging the tires on the friction-test vehicle with the tires on one of the vehicles for which short stopping distances were recorded. The coefficients of friction for the surface on the taxiway were then found to be from 23 to 33 percent higher than with the original tires, as shown in the last column of table 3. The coefficients were then of sufficient magnitude to account for the braking distances recorded during the high-speed tests in which high coefficients of friction were utilized.

Similar differences in friction coefficients between the two sets of tires were found for a bituminous surface, the results for which are included in table 3. It was surprising to find that the tires made such a great difference, especially since both sets of tires had the same tread pattern and were fabricated from the same rubber compound (based on the manufacturer's records of the serial numbers).

The hardness to which the rubber had been cured appeared to be the only measur-

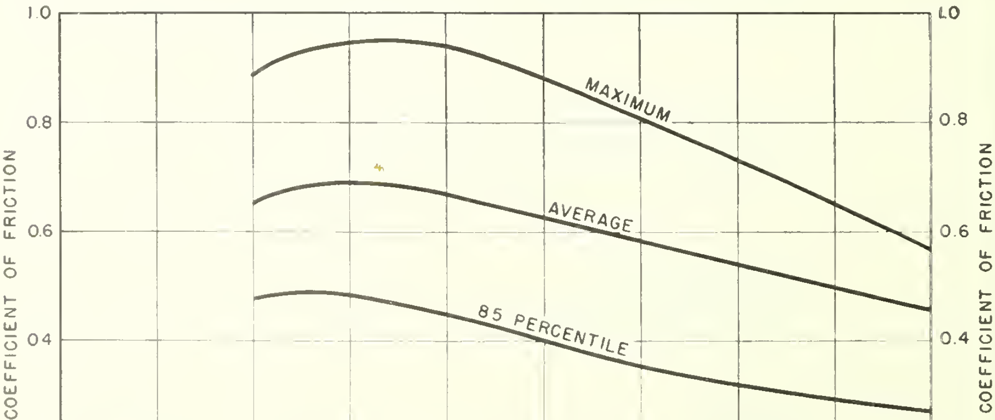

Figure 14.—Average utilized friction coefficient while braking.

Table 3. — Effect of tires on friction coefficients

Type of test	Coefficient of friction	
	First set of tires	Second set of tires
Concrete surface on taxiway:		
Slide impending from stopped position	0.77	0.95
Slide impending with wheels turning at slow speed	.69	92
Sliding at slow speed	.67	.86
Slide impending at 25 miles per hour	.57	....
Sliding at 25 miles per hour	.46	....
Bituminous concrete surface:		
Slide impending at slow speed	.56	.70
Sliding at slow speed	.53	.68

able difference between the physical characteristics of the two sets of tires. Had an attempt been made to find the tires that would result in the highest and lowest friction factors, the difference might well have been much greater than the difference between the two sets that were used.

A few tests were also made on a third set of tires by towing a light pickup truck with a large 10-wheeled wrecker. The resulting coefficients of friction were 53 percent higher than for the first set of tires and 25 percent higher than for the second set of tires. The third set of tires was one size larger and had a different tread pattern than the other two sets of tires. The rubber used in their fabrication may also have been of a different compound and cured to a different hardness than either of the other two sets of tires.

In view of these results, further study should be made to obtain conclusive answers to a number of questions directly related to highway safety. How many drivers realize when they buy a new set of tires that their stopping distances in emergencies may be 30 percent greater with one set of tires than with another set? For safety reasons, would it be equally desirable to reduce the nonuniformity in tires as the nonuniformity in the texture of road surfaces to improve coefficients of friction between tires and road surfaces? Also, in brake-performance tests, to what extent are coefficients of friction between the road surface and the particular set of tires being measured rather than brake performance?

Further Friction Studies

The studies on the taxiway surface were for the purpose of obtaining some idea of the relation between friction coefficients and stopping distances. While the equipment for measuring friction coefficients was available, it appeared desirable to extend the studies to other surfaces, principally to obtain a better idea of the problems involved in programming a comprehensive study of the relation between friction coefficients and braking distances.

To measure friction coefficients of road surfaces, a four-wheeled vehicle was towed with a cable and the towing force measured with a resistance strain-gage dynamometer. This method was selected as the most suitable one available after reviewing other methods reported for previous investigations.

The towed vehicle used for these tests was an Army jeep with a four-wheel drive, new brakes, and passenger-car tires. Two hydraulic shock absorbers were mounted on the front bumper in such a manner that they served as the front support for the tension-bar dynamometer used in conjunction with an electronic strain recorder. The shock absorbers also served to dampen vibrations and variations in the towing force imparted to the dynamometer by the cable when on rough surfaces. The other end of the dynamometer was fastened to the bumper of the jeep with a connection that permitted the same flexibility as a universal joint. The towing force was kept parallel with the road surface at all times by a 25-foot steel cable attached to the differential of the tow truck at the same height as the mounting on the bumper of the jeep. Figure 15 shows the method of connecting the electric dynamometer to the towed vehicle, and figure 16 shows a typical dynamometer recording.

A resistance strain-gage dynamometer and an electronic recorder furnished by the Naval Gun Factory were used throughout these tests. Gages were placed on the four sides of the half-inch-square aluminum bar with a temperature control element. The unit was waterproofed to permit operation in any type of weather. The SR-4 strain recorder was fastened to shock mountings in the tow truck and connected with the dynamometer bar by insulated wires fastened to

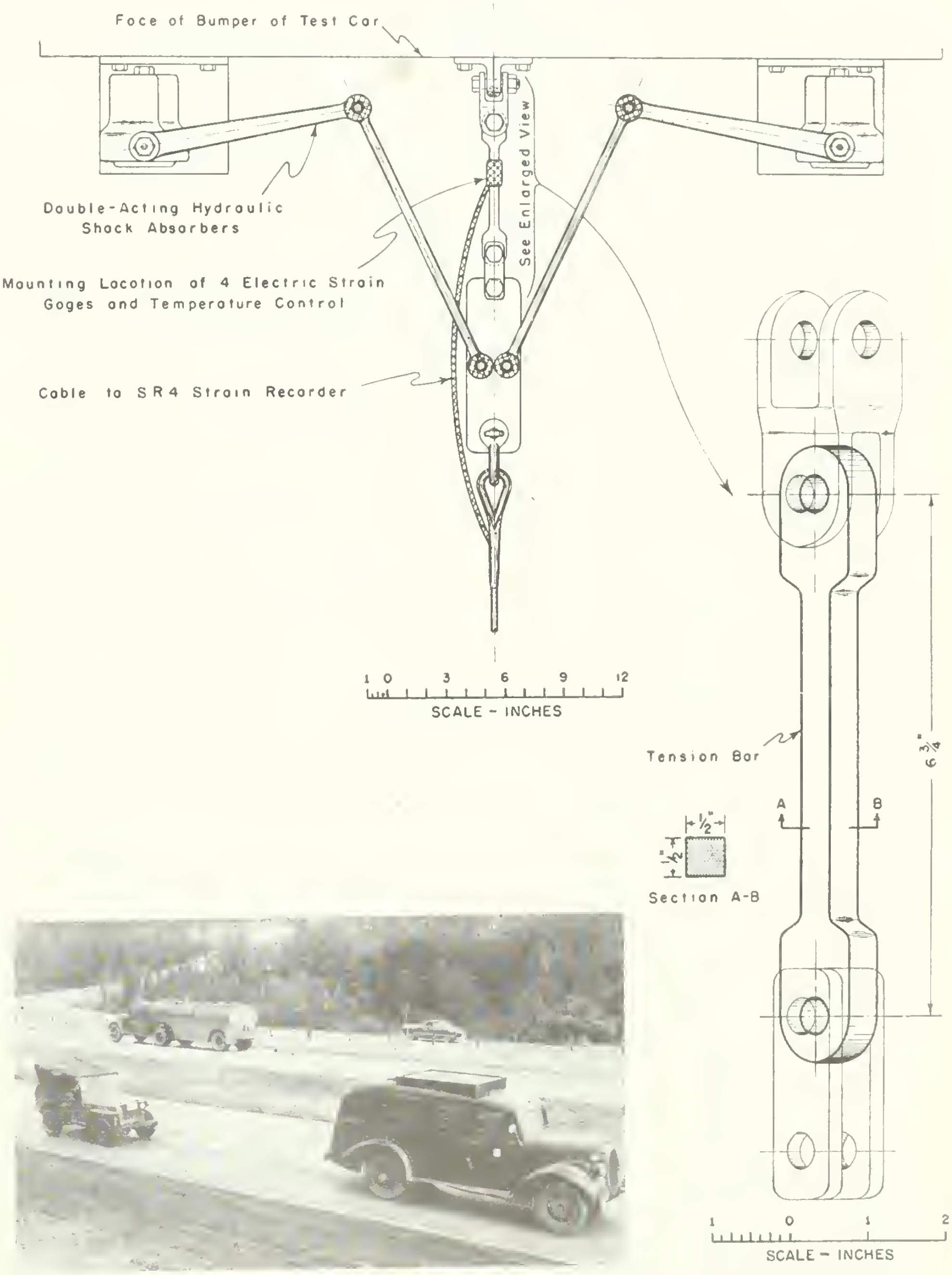
Face of Bumper of Test Car
See Enlarged View
Double-Acting Hydraulic Shock Absorbers
Mounting Location of 4 Electric Strain Gages and Temperature Control
Cable to SR4 Strain Recorder
1 0 3 6 9 12
SCALE - INCHES
Tension Bar
6¾"
A
B
½"
½"
Section A-B
1 0 1 2
SCALE - INCHES

Figure 15.—Method of connecting electric tension dynamometer to towed vehicle. Inset—towing jeep with panel truck.

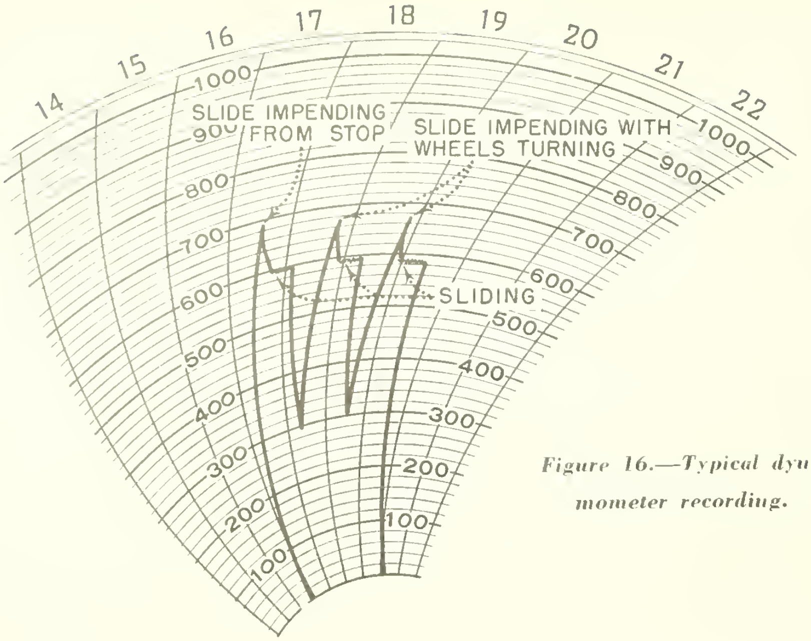

Figure 16.—Typical dynamometer recording.

the tow cable. The calibration of the dynamometer was checked before starting and after completing the friction tests.

The following three types of friction coefficients were measured at each test location:

1. The maximum starting coefficient of friction from a stopped position.
2. The maximum impending coefficient of friction with the wheels turning and a skid impending.
3. The sliding coefficient of friction with the wheels locked.

A standard procedure was followed for each test which involved the following steps (these were repeated at least once at each location to obtain a check on the initial readings):

1. The brakes of the jeep were fully applied with the motor stopped and the transmission in low gear.
2. The tow truck moved ahead in low-low gear, engaging the clutch slowly until the towed vehicle started to slide. The tow truck then continued at a slow speed until the entire test had been completed.
3. After the jeep had moved about 10 feet with its wheels locked, the operator disengaged the clutch and released the brakes on the towed vehicle until the wheels started to turn. He then applied the brakes slowly to obtain the maximum braking force without locking the wheels and then released the clutch slowly. This caused all wheels to start sliding simultaneously.
4. After sliding ahead about 10 feet with the wheels locked, operation 3 was repeated.

Various Surfaces Tested

In addition to the friction tests that were conducted on the taxiway at Andrews Air Force Base, 108 tests were conducted in 1950 on the section of concrete on U S 40 where brake tests had previously been conducted on vehicles selected at random. Friction tests were also conducted at 25 locations selected at random around Washington, D. C., to obtain some idea of the various conditions encountered by drivers in the normal operation of their vehicles. One of these locations was on Memorial Bridge where rear-end collisions frequently caused long delays to traffic crossing the bridge during morning and evening rush hours, especially on days when the surface was wet. The measurements were repeated at 11 of the test locations during a rainy period or between intermittent showers while the surfaces were wet.

The results of these tests show that the taxiway on which the brake tests from high speeds were conducted had a higher coefficient of friction when dry than any of the road surfaces on which tests were made. The taxiway had an average coefficient of friction 5 percent higher than the surface in Maryland where the random brake-performance tests were conducted.

The range in friction coefficients on a 1-mile section of highway on and in the vicinity of the half-mile section where the random brake-performance tests were conducted is shown in figure 17. These were obtained with the set of tires that resulted in the lower coefficients, but the range shown in figure 17 is nevertheless significant.

With a range of more than 10 percent in the coefficient of friction of the surface, depending on the exact location of the test, and a possible variation of at least 30 percent due to the tires, it is evident that these variations had some effect on the results of the brake-performance tests. Was brake performance or the coefficient of friction between the road surface and the tires being measured? It is evident that the results for one must be considered in combination with the other.

The relative coefficients of friction at low speeds for seven different types of surfaces when dry and wet are shown in figure 18. The term "relative" is used because the values are based on one particular set of tires. With other sets of tires the values might be considerably higher or lower and there is not any positive assurance that the relative magnitude of the coefficients between different surfaces would be the same as shown in figure 18. In fact, the results of a limited number of tests on different surfaces with different tires indicate the possibility that one surface with a relatively high coefficient of friction as compared with other surfaces when measured with one set of tires may have relatively a much lower coefficient of friction when the measurements are made with another set of tires. It can be hoped, however, that the data shown in figure 18 do represent about average conditions on a relative basis. They should not be used without this qualification and without the additional qualification that construction methods and other factors can result in a wide variation in friction coefficients even for the same general type of surface.

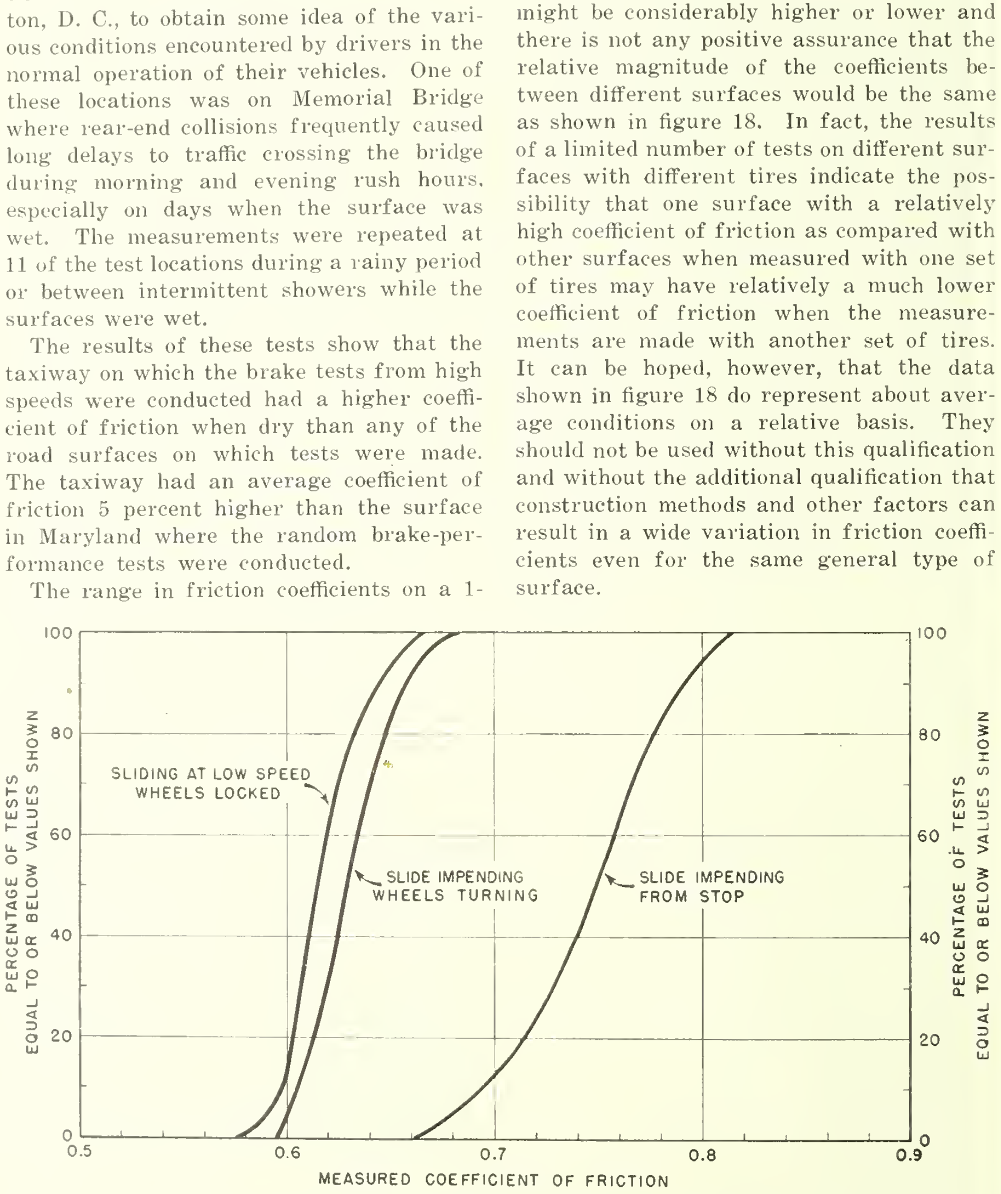

Figure 17.—Range in coefficients of friction on 1-mile section of concrete pavement where random brake-performance tests were conducted.

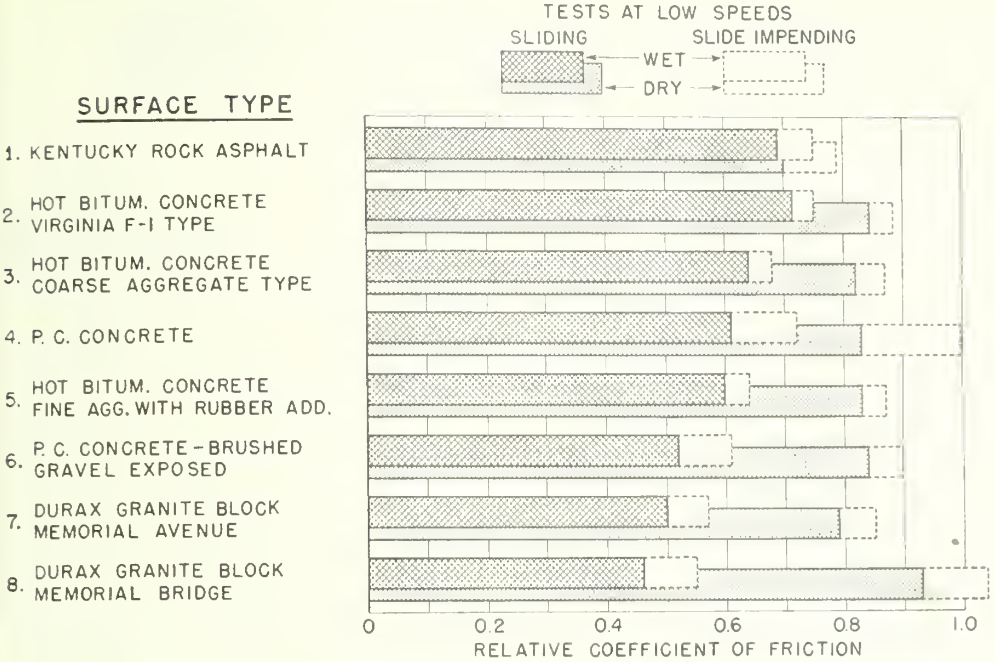

Figure 18.—Relative coefficients of friction (impending coefficient for portland cement concrete used as unity).

Figure 18 shows that the impending coefficients of friction immediately before the wheels start sliding are higher for all types of surfaces, both dry and wet, than the coefficients after the wheels start to slide. Also, all surfaces when wet have lower friction coefficients than when dry.

Interesting Comparisons Made

A most interesting comparison exists between the Durax granite block surface on Memorial Bridge, where heavy traffic volumes had worn the peaks on the blocks smooth, and the same type of surface on Memorial Avenue, where traffic volumes since construction had been much lower than on the bridge. When dry, the worn surface on the bridge had a higher coefficient of friction than any of the road surfaces, but when wet it had the lowest coefficient of friction. Both the impending and sliding values for the wet condition were only about 50 percent of the values for the dry condition. This was undoubtedly an important contributing factor to the large number of rear-end collisions and at least two head-on collisions that occurred while the surface was wet.

When Memorial Bridge was resurfaced with a rock asphalt mixture, the coefficient of friction for the dry condition was reduced about 25 percent. During the more critical condition, however, when the surface was wet, resurfacing the bridge increased the impending coefficient of friction 36 percent and the sliding coefficient of friction 53 percent.

It is also interesting to note that all of the surface materials except Kentucky rock asphalt and the granite blocks have about the same coefficients when dry and a wide range in the coefficients when wet. At one intersection, the intersection area and the approaches for a short distance from the intersection had been resurfaced with a hot bituminous concrete containing a fine aggregate and a rubber addition as an experiment. The resurfaced areas (type 5 in figure 18) actually had a lower friction coefficient than the sections on the approaches that had not been resurfaced (type 2).

The effect of temperature on friction coefficients was studied at 12 locations by conducting tests while the air temperature was 36°F. on one day and repeating the tests on another day when the temperature was 53°. It was cloudy on both days, and there had been a change in air temperature of only 2° during the 24 hours prior to the tests.

The difference in temperature apparently had no effect at five locations with concrete surfaces. At three locations, the coefficients of friction were slightly higher, and at two locations they were slightly lower at 53° than at 36°. At the seven locations with various types of bituminous surfaces, the friction coefficients were consistently higher at the lower temperature than at the higher temperature. The average difference was 10 percent for the impending coefficients of friction with the wheels turning and 8 percent with the wheels sliding in a locked position.

Need for Further Research

The tests to determine braking distances of vehicles from high speeds and the tests of friction coefficients may be regarded as pilot studies pointing to the need for far more extensive studies that should involve the cooperation of the automotive and tire industries. The brake-performance tests may seem to be of limited number but, as far as is known, they have not been made elsewhere in larger numbers.

The tests were of sufficient scope to throw serious doubt on some of the beliefs and opinions accepted in the past and to suggest the need for research broad enough in scope to give conclusive answers to the following questions:

1. What is the braking force or deceleration rate that drivers should be expected to attain on a dry surface? The tests showed that drivers of passenger cars with brakes in good condition were not always capable of obtaining a braking force sufficient to lock the wheels on dry surfaces with high coefficients of friction, especially at high speeds.

2. Are not distances within which most vehicles can be stopped from high speeds most of the time on surfaces with high coefficients of friction considerably longer than those generally accepted to be correct in the past? In these limited tests vehicle braking distances on dry surfaces increased with increased speed at a rate greater than the square of the speed.

3. What causes the large variation in the braking distances of different vehicles and of the same vehicle during successive stops? During these tests there was a wide variation in the braking action of the same vehicle and in the braking action of different vehicles, especially at high speeds.

4. To what extent do brakes and road conditions cause vehicles to dive to one side and the drivers to lose control of their vehicles? In these tests deceleration rates attained by vehicles in making stops from high speeds were not uncomfortable unless the wheels locked or the driver was unable to control the path of the vehicle due to improperly adjusted brakes or nonuniformity of the road surface.

5. What is the exact extent to which brake fade and other factors affect stopping distances of vehicles under normal operating conditions? In these tests brake fade appeared to be one of the most serious deficiencies of the cars tested.

The results of the friction coefficient measurements are useful principally as a pilot study to illustrate the necessary magnitude of any investigation designed to obtain exact information on the interrelation of stopping distances and friction coefficients between tires and road surfaces. Much is being done by highway departments to improve the nonskid qualities of roadway surfaces and to eliminate types that are exceedingly slippery when wet. Is it not also necessary to consider the variation in tires concurrently with the road surfaces to improve friction coefficients? Improvement in operating safety can be expected from continued attention to better road surface design and from continued improvement in the construction of tires. Would it not be advisable to establish minimum standards for both road surfaces and tires to avoid having drivers confronted with unexpectedly hazardous friction factors on both wet and dry surfaces?

The Interstate Highway Accident Study

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
MORTON S. RAFF
Mathematician

While highway designers often talk about building safety into the highways, there have been few comprehensive studies of just what it is that makes some roads safer than others. In the study reported here, more than 16,000 accidents on approximately 5,000 miles of highway in 15 States have been used to relate the accident rates on different sections of highway to their respective design features and traffic characteristics. The roads selected are all main rural highways.

The most significant factors affecting accident rates are the number of lanes, the volume of traffic, the degree of curvature, the widths of the pavement and the shoulders, and the percentage of cross traffic at intersections. Most of the effects are in the expected directions, but there are certain exceptions.

At high traffic volumes, the lowest accident rates are to be found on divided roads with controlled access, while the highest rates occur on three-lane roads. On most types of highway sections the accident rate becomes higher with increasing traffic volume, except for a slight reversal due to congestion at extremely high volumes. However, there is a different pattern for two-lane curves and intersections, where the accident rate declines as the volume increases.

Sharp curves have higher accident rates than flat curves on roads carrying the same amount of traffic. Wide pavements and shoulders are conducive to safety on two-lane curves, though they do not appear to have any value on two-lane tangents. Bridges are greatly helped by having the bridge roadway several feet wider than the approach pavement.

At intersections, the percentage of the total traffic which is on the minor road is extremely important. Intersections where the cross traffic is between 10 and 20 percent of the total have more than twice the accident rate of intersections having less than 10 percent cross traffic. Also, three-way intersections are considerably safer than four-way crossings.

A number of roadway features failed to show any consistent relation to accident rates. These include grade, the frequencies of curves and sight restrictions, and the percentages of commercial and night traffic.

A POPULAR VIEW has it that every accident results from some "principal cause," like speeding or driving on the wrong side of the road. The way to prevent accidents, according to this view, is to stop drivers from doing the things that stand highest on the list of principal causes.

The problem is not so simple. Every accident has many causes, if we consider a cause to be any remediable condition whose correction would have prevented the accident. For example, suppose two cars driven at high speed have a head-on collision at night on a two-lane road. The speed of the vehicles is obviously one cause of the collision. Other causes may be the use of blinding headlights, the absence of lighting on the highway, inadequate pavement width, and the fact that the road carries two-way instead of one-way traffic. Any of these may have contributed equally with speed to the accident, but the chances are that speeding will get most of the blame.

It is desirable to examine all the causes of an accident instead of concentrating on a single cause, since a wide variety of corrective measures may be suggested by a broad approach. Classifying accidents according to all their circumstances, regardless of how unimportant any particular circumstance may seem at first, offers the hope of discovering significant relations between accident frequencies and associated circumstances which might otherwise escape notice.

The present study is an attempt to find out how rural traffic accident rates are affected by various physical features of the highway and by certain use characteristics such as average daily traffic, percentage of commercial vehicles, and the like. These are by no means the only causes of accidents. But if it should turn out, for instance, that roads with flat curves are appreciably safer than roads with sharp curves, then it would be possible to predict with some assurance the accident savings that would result from building flatter curves into the highways. Accidents have many causes, and an effective accident-reduction program ought to use the full range of remedies. This study is intended to throw light on those remedies which are in the domain of the highway designer.

Summary of Findings

The most significant factors affecting accident rates, as discovered in this study, are

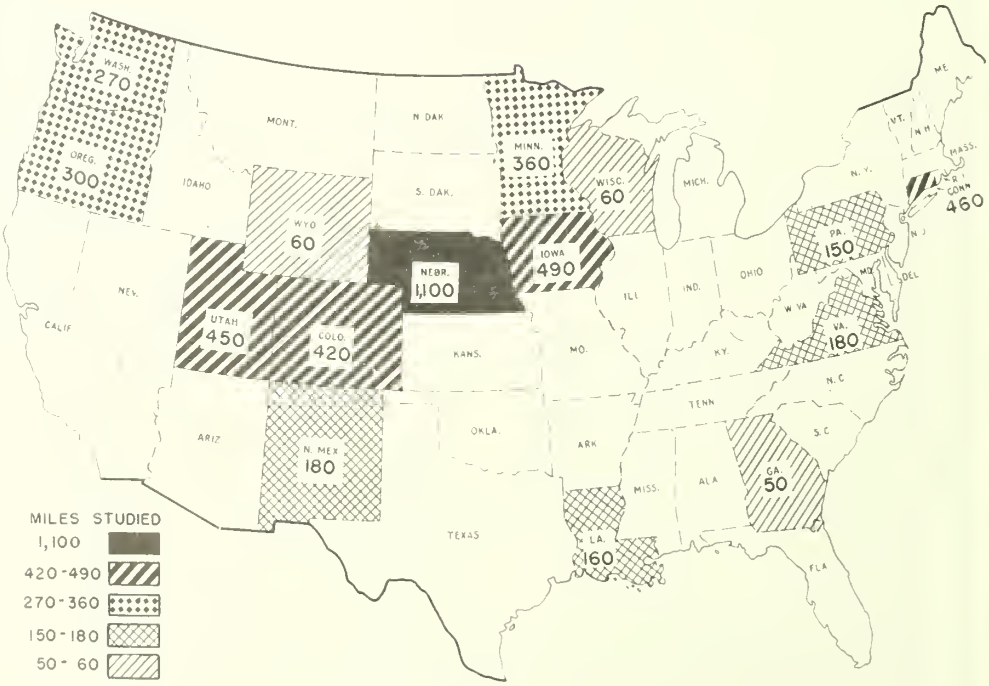

Figure 1.—Mileage of study routes.

Table 1.—Length and amount of travel on the study routes

State	Study year	Approximate length, in miles: Tangents	Approximate length, in miles: Curves	Approximate length, in miles: Total	Travel in study year in million vehicle-miles: Tangents	Travel in study year in million vehicle-miles: Curves	Travel in study year in million vehicle-miles: Total
Colorado	1941	350	70	420	380.4	77.3	457.7
Connecticut	1941	300	150	450	639.1	249.0	888.1
Connecticut	1946	310	150	460	508.6	200.0	708.6
Georgia	1940	40	10	50	40.5	13.7	54.2
Iowa	1941	430	60	490	303.2	41.9	345.1
Louisiana	1941	130	30	160	175.0	30.9	205.9
Minnesota	1946	290	70	360	180.4	48.0	228.4
Nebraska	1941	1,000	100	1,100	547.5	57.3	604.8
New Mexico	1941	160	20	180	64.9	5.0	69.9
Oregon	1941	210	90	300	220.9	58.0	278.9
Pennsylvania	1941	100	50	150	133.2	74.0	207.2
Utah	1941	370	80	450	238.9	46.0	284.9
Virginia	1941	150	30	180	275.4	54.9	330.3
Washington	1941	200	70	270	469.1	135.8	604.9
Wisconsin	1941	50	10	60	70.3	11.0	81.3
Wyoming	1941	50	10	60	22.4	4.4	26.8
Total		4,140	1,000	5,140	4,269.8	1,107.2	5,377.0

traffic volume, degree of curvature, pavement and shoulder width on curves, percentage of cross traffic at intersections, and the width of bridge roadways, both absolutely and in relation to their approach pavements. In most cases the effects are in the expected directions, but there are certain exceptions.

Volume of traffic has a strong effect on the accident rate on nearly all types of highway sections. In general—except for curves and intersections on two-lane roads—the accident rate becomes higher as the volume is increased. There is often a slight reversal of this trend at very high volumes, presumably because extreme congestion inhibits the drivers' ability to make passing maneuvers.

At curves and intersections on two-lane roads the trend goes the other way. Here the accident rates become lower with increased traffic volume. This effect has been well substantiated, but the reason for it remains a matter of speculation. A plausible theory is that the two-lane curves and intersections present conditions which most drivers recognize as hazardous, particularly when there is a considerable amount of traffic. Accordingly, the driver pays enough extra attention when these facilities are busy to more than compensate for the added potential danger.

Sharp curves have higher rates than flat curves. The volume effect described above is more pronounced on sharp curves than on flat ones.

Wide pavements and shoulders help to reduce the accident rates on two-lane curves. This is in contrast to the two-lane tangents, where no particular effect could be traced to the width of the pavement or the shoulders.

The percentage of cross traffic at an intersection has a tremendous effect on its accident rate. It takes only about 15 percent cross traffic to make an intersection more than twice as hazardous as when the cross traffic is well below 10 percent.

Another important intersection characteristic is the number of approaches. Three-way intersections (T and Y) have markedly lower accident rates than four-way crossings. This is not necessarily an argument for staggering all crossings, however, as the increased number of intersections and the additional turning and weaving might easily nullify the apparent advantage.

Wide roadways are desirable at two-lane bridges and underpasses, and they should be several feet wider than the approach pavements. Of the two types of structures, underpasses are considerably more hazardous.

A number of roadway features did not appear to have any consistent effect on the accident rates. These include grade, pavement and shoulder widths, frequency of curves, frequency of sight restrictions, and the percentages of commercial and night traffic.

Technique of Study

To make it possible to study a large number of different highway features, the roads included in the study were divided into short homogeneous sections. Each of these sections was substantially uniform in grade, pavement width, shoulder width, degree of curvature, traffic volume, etc. Any place where a change occurred in any of these characteristics was made a dividing line between one section and another. Intersections and structures were also regarded as sections, because of their special characteristics

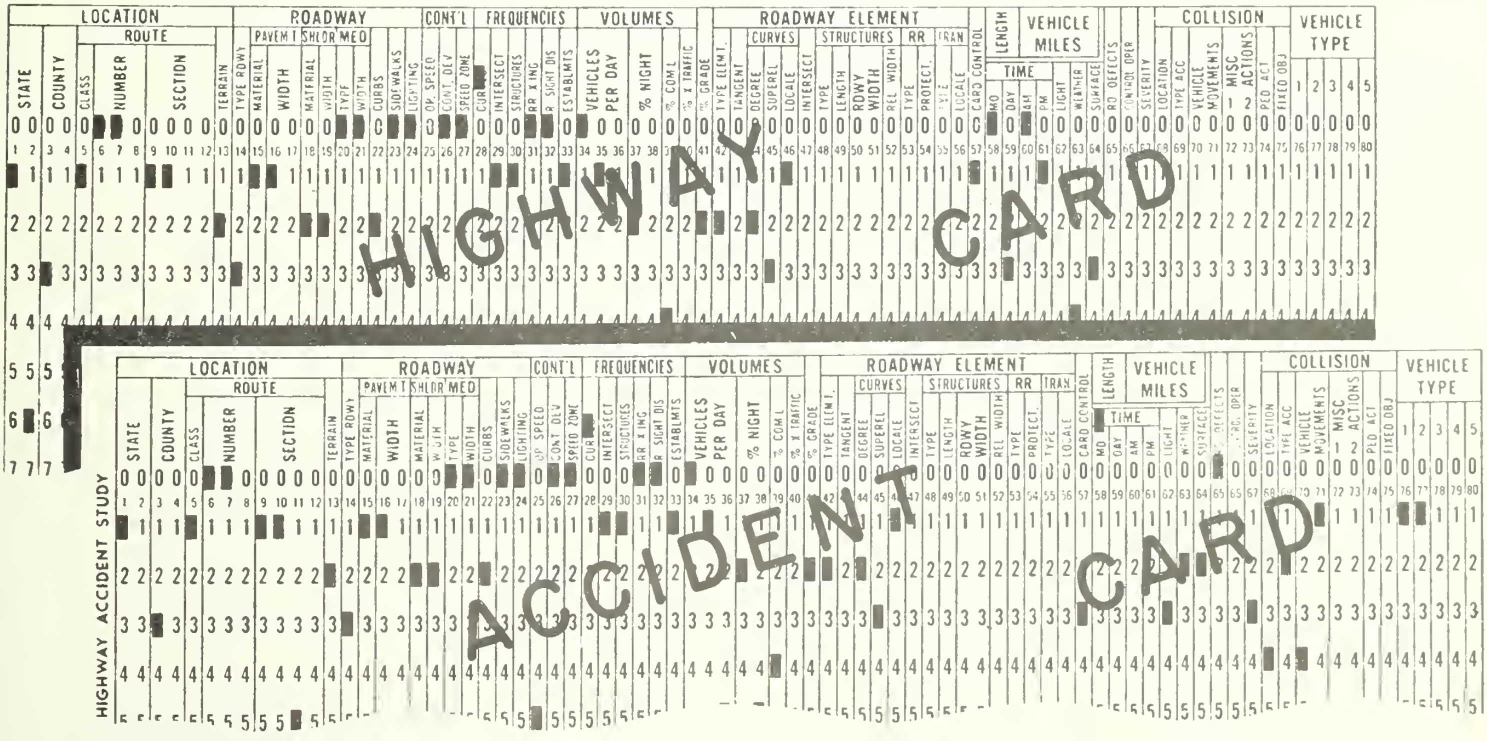

Figure 2.—A highway punch card and an accident punch card on the same highway section.

Table 2.—Number of highway sections, by type of roadway element

State	Tangents	Curves	Inter-sections	Structures	Railroad crossings	Other	Total
Colorado	1,239	571	604	207	13	2	2,636
Connecticut (1941)	1,634	1,356	1,027	311	9	106	4,443
Connecticut (1946)	1,628	1,317	1,020	329	9	106	4,409
Georgia	160	84	16	6	0	0	266
Iowa	1,434	493	686	293	13	0	2,919
Louisiana	333	148	108	64	19	11	683
Minnesota	1,402	533	612	54	13	2	2,616
Nebraska	2,683	559	1,107	298	34	0	4,681
New Mexico	416	77	16	58	0	0	567
Oregon	626	771	322	122	9	0	1,850
Pennsylvania	338	239	12	203	0	11	803
Utah	896	530	276	51	18	27	1,798
Virginia	832	240	263	92	2	14	1,443
Washington	1,093	610	537	83	17	0	2,340
Wisconsin	232	69	91	16	2	2	412
Wyoming	132	58	30	4	1	0	225
Total	15,078	7,655	6,727	2,191	159	281	32,091

Table 3.—Number of accidents, by type of roadway element

State	Tangents	Curves	Inter-sections	Structures	Railroad crossings	Other	Total
Colorado	676	130	225	80	0	0	1,111
Connecticut (1941)	1,374	560	426	11	0	16	2,387
Connecticut (1946)	1,223	439	287	81	1	67	2,098
Georgia	64	32	17	8	0	0	121
Iowa	461	100	98	20	1	0	680
Louisiana	531	39	107	14	2	0	693
Minnesota	254	46	170	13	5	0	488
Nebraska	638	77	121	29	6	0	871
New Mexico	89	10	9	1	0	0	109
Oregon	926	106	366	118	0	0	1,516
Pennsylvania	265	140	22	19	0	55	501
Utah	447	120	116	16	7	0	706
Virginia	648	164	250	51	0	2	1,115
Washington	1,962	509	1,119	86	17	1	3,694
Wisconsin	197	7	68	2	0	0	274
Wyoming	45	4	8	0	0	0	57
Total	9,800	2,483	3,409	549	39	141	16,421

Table 4.—Number of accidents, by severity, and ratios and adjustment factors

State	Number of accidents				Ratio of—			
	Fatal	Injury	Other	Total	Total to fatal	Total to fatal-plus-injury	Injury to fatal	Adjustment factor [1]
Colorado	46	382	683	1,111	24.2	2.60	8.3	2.07
Connecticut (1941)	51	1,018	1,318	2,387	[2] 46.8	2.23	20.0	1.07
Connecticut (1946)	42	758	1,298	2,098	[2] 50.0	2.62	18.0	1.00
Georgia	8	47	66	121	15.1	2.20	5.9	3.31
Iowa	30	274	376	680	22.7	2.24	9.1	2.20
Louisiana	22	260	411	693	[2] 31.5	2.46	11.8	1.59
Minnesota	10	178	300	488	[2] 48.8	2.60	17.8	1.02
Nebraska	46	406	419	871	18.9	1.93	8.8	2.65
New Mexico	11	59	39	109	9.9	1.56	5.4	5.05
Oregon	39	288	1,189	1,516	[2] 38.9	4.64	7.4	1.29
Pennsylvania	20	193	288	501	[2] 25.0	2.35	9.7	2.00
Utah	33	283	390	706	21.4	2.23	8.6	2.34
Virginia	102	434	579	1,115	10.9	2.24	4.3	4.59
Washington	84	975	2,635	3,694	[2] 44.0	3.49	11.6	1.14
Wisconsin	9	89	176	274	[2] 30.4	2.80	9.9	1.64
Wyoming	4	27	26	57	14.2	1.84	6.8	3.52
Total	557	5,671	10,193	16,421	29.5	2.64	10.2	....

[1] 50 divided by the total-to-fatal ratio. These adjustment factors are used in computing the type 1 accident rates.
[2] Type 2 accident rates are computed only for those States with total-to-fatal ratios of at least 25.

(e.g., volume of traffic on the intersecting road, relative width of bridge roadway and adjoining pavement). The presence of an intersection or a structure was treated as a break between highway sections. Each accident was assigned to the highway section where it occurred.

The basic techniques for the study were devised by the National Safety Council in cooperation with the Bureau of Public Roads in 1945, following a pilot study on U S 1 in Virginia. All 48 States were invited to participate in the study, using as data the 1941 accidents on rural sections of the National System of Interstate Highways. Only 15 States were able to do so, although a number of others expressed interest in the project. The year 1941 was selected as the most recent in which driving conditions were "normal." A few of the States used other years (see table 1), and some main rural highways were used which are not part of the National System of Interstate Highways. The chief obstacles to wider participation by the State highway departments were insufficient manpower to prepare the strip maps and the coding sheets, inability to locate accidents accurately, and incomplete accident reporting, in the sense that too many accidents went unreported. Two brief reports of preliminary findings have already been published.[1]

The participating States are given in figure 1 and table 1. For each State the table gives the year for which accident records were submitted, the number of miles of roadway included in the study, and the total amount of travel on those roads during the study year. (In addition to 1941 data, Pennsylvania submitted cards for accidents in the first 5 months of 1942, but these have not been used in any of the analyses covered in this report.)

The data were recorded on tabulating cards, the coding procedure calling for two sets. One set, called highway cards, contains a card for each highway section. The second set, called accident cards, contains a card for each accident. In figure 2, the upper picture is a typical highway card; below it is an accident card representing an accident on the same highway section. The first 56 columns, which identify and describe the highway section, are identical on the two cards. This makes it possible to classify accidents according to various highway features without having to refer to the highway cards.

The punch in column 57 indicates which type of card it is. The remaining columns serve different purposes on the two types of cards. On a highway card they give the length of the section and the annual vehicle-mileage of travel on it. On an accident card these columns contain information about the circumstances of the accident.

A third type of card, the summary card, has recently been punched and used in some of the later analyses. There is one of these cards for each highway section, with the columns at the end of the card containing information about the number of accidents on the section.

Number of Accidents

Tables 2–4 indicate the size of the study. Table 2 shows the number of highway sections in each State, subdivided by types of roadway elements. Nearly half of the 32,091 highway cards represent tangent sections, and one-fourth represent curve sections. About two-thirds of the remainder (21 percent of the total) represent intersections, and the rest are for structures, railroad crossings, toll stations, and transitions of various kinds.

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[1] *A plan for relating traffic accidents to highway elements*, by C. F. McCormack. American Association of State Highway Officials Convention Group Meetings, 1944, pp. 117-119. *The relation of highway design to traffic accident experience*, by D. M. Baldwin. American Association of State Highway Officials Convention Group Meetings, 1946, pp. 103-109.

Table 3 shows similar information regarding the accident cards. In all, there were 16,421 accidents, of which 60 percent were on tangents and 16 percent on curves.

Table 4 classifies the cards by severity of accident within each State. There are cards for 557 fatal accidents, 5,671 personal-injury accidents, and 10,193 property-damage accidents.

Table 4 also lists certain ratios for each State. The ratio of the total number of accidents to the number of fatal accidents is a rough measure of the completeness of accident reporting. It has its limitations, however, for while all fatal accidents are probably reported, it is most unlikely that the true total-to-fatal ratio is the same in every State. In any event, this ratio is the basis of an adjustment which is used in some of the rate computations.

The ratio of all accidents to those involving either deaths or injuries (fatal-plus-injury) has a similar interest. It might also be used for adjustments, but this has not been done so far. This ratio varies much less among the different States than does the total-to-fatal ratio. The table also shows the ratio of the number of injury accidents to the number of fatal accidents in each State, and the last column of table 4 lists the adjustment factors (obtained by dividing 50 by the total-to-fatal ratios) which are used in computing the type 1 accident rates, as explained subsequently.

Method of Analysis

It was difficult to decide how to combine the detailed data from different States. The reporting requirements vary, and it cannot be assumed that the reporting laws are fully complied with in every State. There are three essentially different ways of dealing with this problem. One way is to use a system of weights involving an adjustment factor for each State. A State which is believed to report only half its accidents would have an adjustment factor of 2, so that each reported accident would count as two adjusted accidents. This is substantially the approach that was used by McCormack and Baldwin in their earlier reports of preliminary findings from this study. The trouble with adjustments is that they give the most weight to the least reliable data, and that the adjustment factors are computed on the basis of a dubious assumption.

A second approach would dispense with adjustments but would use only the data from States whose reporting meets a certain standard. This avoids the distortions caused by the adjustment process, but it has the drawback of reducing the amount of usable data. A variation of this approach would use only the fatal accidents (in all the States), or only the fatal and injury accidents. To use only fatal accidents is impractical, however, for it would reduce the study to only 557 accidents. The use of both fatal and injury accidents has more to commend it, but there would still be a heavy reduction in the amount of data available for analysis. Moreover, it is doubtful that fatal (or fatal and injury) accidents are affected by highway features in the same way as accidents of all degrees of severity combined.

The third approach is to ignore the problem altogether and simply count the accidents in all the participating States, irrespective of the variation in reporting standards. For all its crudeness, this method turns out to be generally superior to the other two.

All three of these approaches have been used. (The choice among them is explained in the final section of the report.) In the ensuing discussion they will be called type 1 rates, type 2 rates, and type 3 rates, respectively. The type 1 rates use all the participating States, with the number of accidents in each State multiplied by the adjustment factor given in the last column of table 4. The type 2 rates do not use adjustments, and include only those States having a total-to-fatal ratio of at least 25. The type 3 rates use all the participating States, without adjustment. Use of the adjustment factors involves the assumption that the total-to-fatal ratio would be the same in every State if the reporting standards were identical. There is at least one State in the present study for which this assumption is clearly unreasonable (see discussion of Virginia, p. 177).

Once it has been decided which material to use and whether the count should be of actual or of adjusted accidents, the process of computing a rate involves (1) counting the total number of accidents on sections in a particular category, (2) adding up the total vehicle-mileage on these sections, and (3) dividing the former sum by the latter to get the rate in terms of accidents per million vehicle-miles. To study the effect of grade, for example, the highway sections are divided into several groups on the basis of their grades. Then the rates for these groups are computed and examined to see if there is a steady trend or other close relation between accident rate and grade. If the rates show indications of a trend but are somewhat irregular, it is possible to make a statistical test of whether or not a trend really exists.

The slope of the "best" straight line—the regression coefficient—is computed, with each rate weighted in proportion to the amount of travel on which it is based. Also computed are the confidence limits of this slope, from which one can tell at a glance how reliable the estimated slope is and whether or not it is significantly different from zero. These quantities are presented in those cases where they will aid in understanding the analyses.

For those to whom the correlation coefficient is more familiar than the regression

Table 5.—Accident rates on tangents, by grade and roadway type

[*rates are per million vehicle-miles*]

Grade, in percent	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided[1]		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
Less than 3	5,442	3.7	194	6.1	1,043	6.0	827	4.7	504	2.2
3–3.99	321	4.0	18	5.6	136	7.1	83	3.4	139	2.9
4–4.99	253	3.6	6	7.7	65	6.8	56	4.4	59	1.6
5–5.99	322	4.4	3	4.4	49	6.8	9	3.6	46	2.1
6–6.99	86	4.0	1	10.0	29	10.2	1	1.4	13	1.7
7 or more	49	3.9	5	14.4	26	10.1	6	15.6	13	1.5
Less than 3	5,442	3.7	194	6.1	1,043	6.0	827	4.7	504	2.2
3 or more	1,031	4.0	33	6.8	305	7.5	155	4.1	270	2.3
TYPE 2 ACCIDENT RATES (SELECTED STATES,[2] WITHOUT ADJUSTMENT)										
Less than 3	3,507	3.0	100	6.5	644	3.3	701	3.0	504	1.6
3–3.99	219	3.3	14	3.4	78	3.5	78	2.5	139	1.7
4–4.99	175	2.7	1	1.2	12	2.0	43	3.0	59	1.5
5–5.99	259	4.2	0	......	18	3.7	6	1.8	46	1.4
6–6.99	50	2.9	0		0	......	1	2.5	13	1.6
7 or more	48	3.7	0		5	6.3	0	....	13	1.5
Less than 3	3,507	3.0	100	6.5	644	3.3	701	3.0	504	1.6
3 or more	751	3.4	15	2.7	113	3.4	128	2.6	270	1.6
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
Less than 3	5,442	2.2	194	2.6	1,043	2.7	827	2.9	504	1.6
3–3.99	321	2.3	18	2.5	136	2.8	83	2.5	139	1.7
4–4.99	253	2.2	6	2.3	65	2.0	56	2.6	59	1.5
5–5.99	322	3.1	3	.9	49	2.3	9	1.8	46	1.4
6–6.99	86	2.2	1	2.0	29	2.4	1	1.4	13	1.6
7 or more	49	3.7	5	3.1	26	2.6	6	3.3	13	1.5
Less than 3	5,442	2.2	194	2.6	1,043	2.7	827	2.9	504	1.6
3 or more	1,031	2.5	33	2.2	305	2.4	155	2.5	270	1.6

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[1] Excluding highways with controlled access. This applies to all the tables.

[2] States having a total-to-fatal ratio of 25 or more. This applies to all the tables.

coefficient, it should be pointed out that the two are closely related. ($b = r\sigma_y/\sigma_x$, where b is the regression coefficient, r the correlation coefficient, and σ_y and σ_x the standard deviations of y and x respectively.) The significance of the departure of b from zero is the same as that for r. The use of b has two advantages, however, over the use of r: (1) it is of more inherent interest, since it may be more important to know about a relationship of steep slope with low reliability than one of small slope with high reliability; (2) if the true relationship is a straight line, the estimated value of b is normally distributed while that of r is highly skewed. So the estimate of b is much less affected by sample size than that of r.

For study purposes the highway sections have been classified as tangents, curves, intersections, structures, and miscellaneous (railroad grade crossings, toll stations, and transitions in width of roadway or median). These are further subdivided according to the number of lanes on the study route.

Some of the material is presented in tables, some in bar graphs. Most of the tables present the three types of accident rates already described. As a guide to the reliability of the various rates, there is presented along with each rate the actual number of accidents on which it is based.

The graphs, with one exception, show only the type 3 rates, i.e., the ones that use accidents from all the States without any adjustments. On these graphs the bars differ in width according to the number of vehicle-miles on which each rate is based.

Tangents: Effect of Grade

Two- and three-lane roads

Table 5 shows how the accident rates on tangents vary with the gradient of the

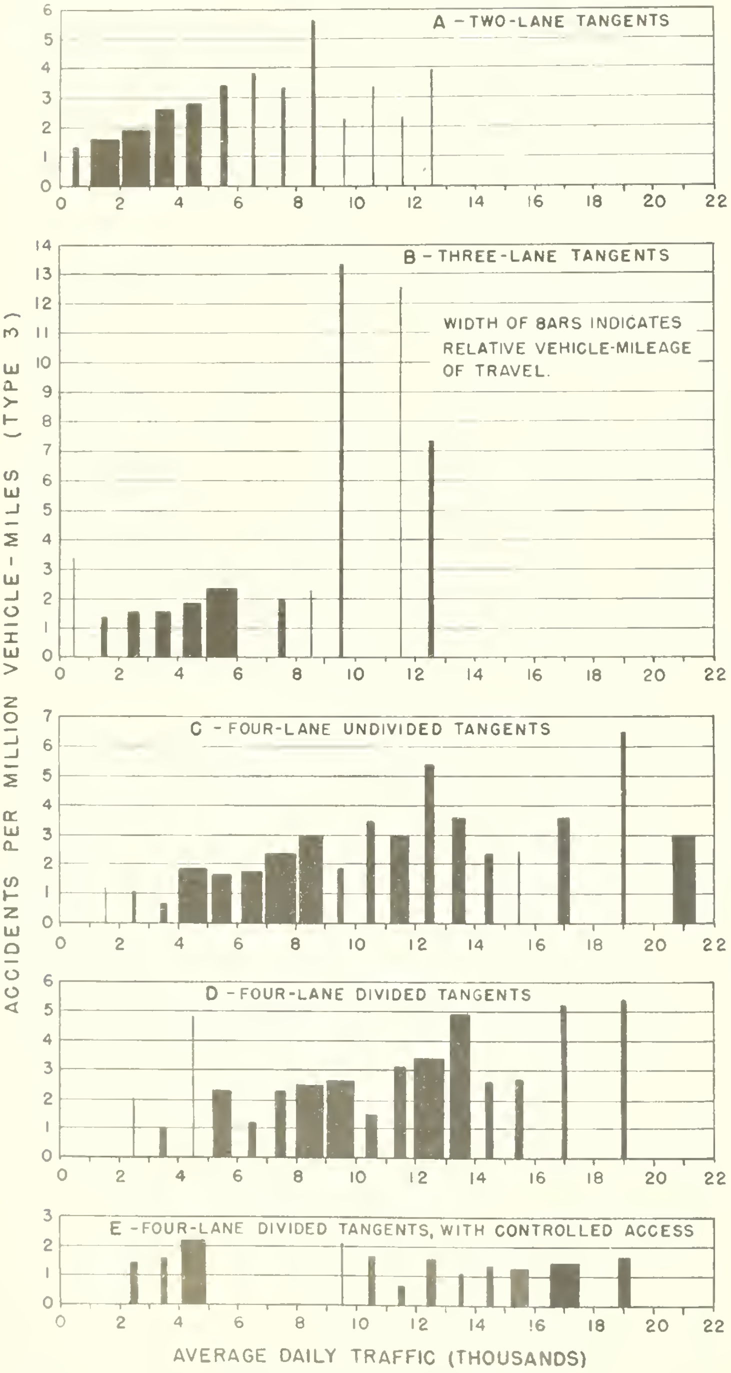

Figure 3 (above).—*Accident rates on tangents, by volume of traffic.*

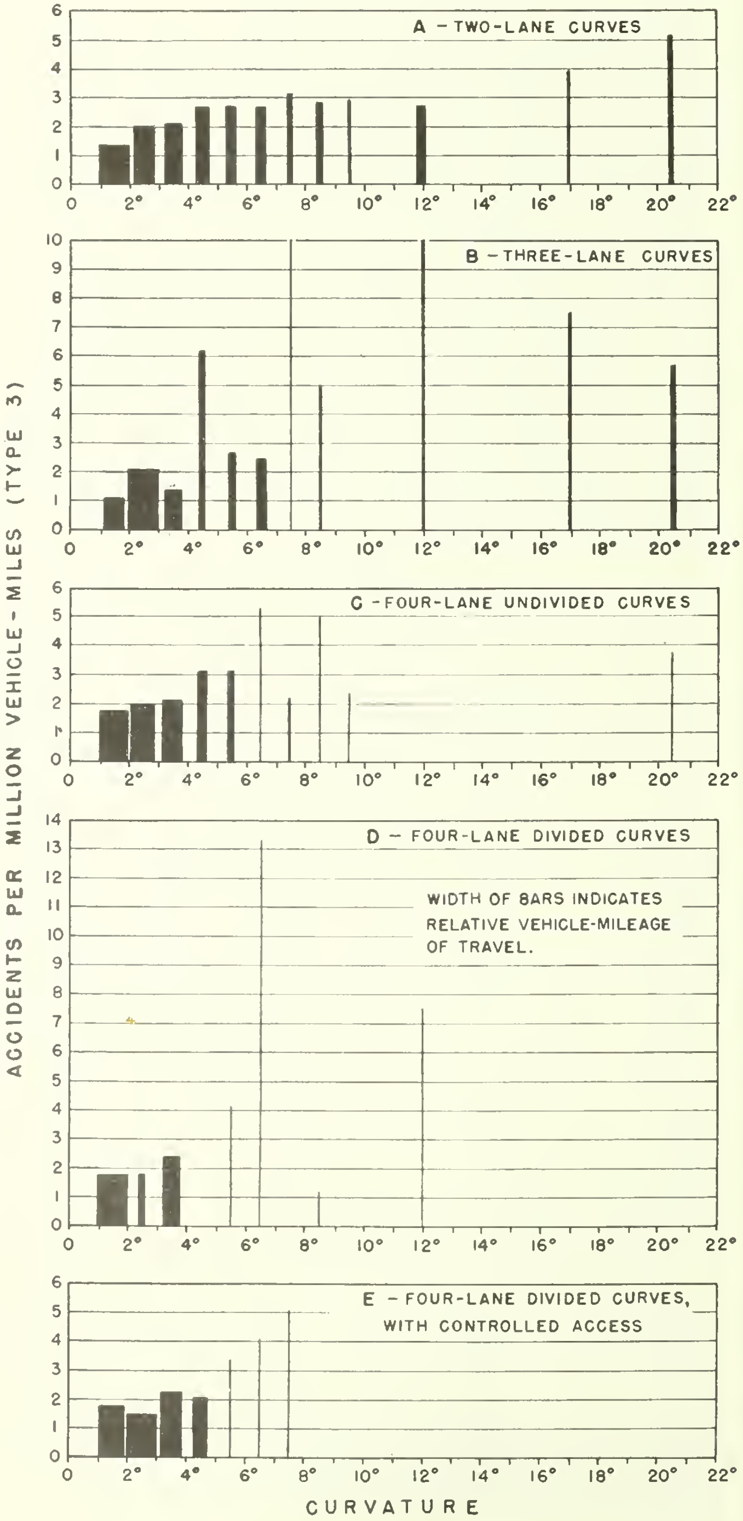

Figure 4 (right).—*Accident rates on curves, by degree of curvature.*

Table 6.—Accident rates on tangents, by volume of traffic and roadway type

[*rates are per million vehicle-miles*]

Average daily traffic	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
0–4,900	5,007	3.6	79	5.3	129	5.6	25	3.1	265	4.0
5,000–9,900	1,396	4.3	102	6.6	481	7.3	388	4.0	3	2.1
10,000–14,900	71	3.6	46	10.4	422	6.9	465	5.3	166	1.4
15,000 or more	0	......	0	....	317	4.1	126	5.1	340	1.5
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
0–4,900	2,868	2.9	5	2.0	18	1.4	19	1.4	265	2.0
5,000–9,900	1,320	3.8	64	4.8	117	3.2	280	2.4	3	2.1
10,000–14,900	71	3.3	46	8.1	309	3.5	380	3.5	166	1.4
15,000 or more	0	....	0	....	314	3.7	126	4.4	340	1.5
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
0–4,900	5,007	2.1	79	1.6	129	1.6	25	1.6	265	2.0
5,000–9,900	1,396	3.6	102	2.9	481	2.2	388	2.4	3	2.1
10,000–14,900	71	3.3	46	8.1	422	3.5	465	3.4	166	1.4
15,000 or more	0	......	0	....	317	3.6	126	4.4	340	1.5

Table 7.—Accident rates on two-lane tangents, by width of pavement

[*rates are per million vehicle-miles*]

Pavement width, in feet	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
16 or less	246	5.5	246	3.3	246	4.3
18	1,795	4.0	742	2.9	1,795	2.1
20	3,263	3.4	2,434	3.0	3,263	2.3
21 or 22	506	4.7	359	3.1	506	2.6
23 or 24	299	3.8	138	3.9	299	1.9
25	45	2.4	38	1.9	45	1.6
26	47	4.0	47	3.6	47	3.3
27	47	3.3	42	3.4	47	2.5
28	132	4.6	132	3.6	132	3.6
29 or more	94	3.1	81	2.8	94	2.5

highway. On two-lane roads there does not appear to be any particular relation between accident rate and grade, no matter which of the three rate types is considered. (This statement refers to the *total* correlation between accident rate and grade, ignoring all other highway features. It may be that, when the appropriate other features are held constant, there is a significant *partial* correlation. All the statements which will be made in connection with single-factor analyses refer to total correlations only.)

For each of the three types the slope (regression coefficient) is positive, i.e., the accident rate tends, on the average, to increase as the grade increases. However, none of the three slopes is significantly different from zero at the 5-percent level of significance. (The 5-percent level is widely used in statistical analyses.) This means that the amount of scatter is such that there is more than a 5-percent chance that the true slope may be zero or negative.

The three types of accident rates differ considerably. The type 1 rate is the highest, because it includes accidents which are assumed to have occurred without being reported. The type 3 rate is the lowest, because it uses only the accidents that were actually reported and includes States in which the reporting is known to be poor. The type 2 rate is in between. It might be thought that this rate would be the most reliable, because it includes only the accidents actually reported in States where reporting is presumed to be good; but it suffers from being based on a smaller sample than the other two types of rates.

A disturbing feature in all three types of rates is the large amount of apparently meaningless fluctuation. For example, the type 2 rate takes the values 3.0, 3.3, 2.7, 4.2, 2.9, and 3.7 as the grades increase steadily. These fluctuations are too large to be due to sampling variation. They cannot be a result of the adjustment process, for they are just as prominent in the unadjusted rates (types 2 and 3) as in the adjusted one; anyway, the same effect is found within individual States. Nor are they peculiar to the effect of grade; similar fluctuations occur with most other highway features. They may be due to the oversimplification caused by studying only one or two features at a time while ignoring all the rest, or they may be a result of the difficulty in obtaining absolutely accurate data. This could have the effect of obscuring relations which really exist. There is some evidence to support the latter belief. After submitting the data for the present study, Minnesota conducted another study of the same highway which in some respects parallels the work being described here. The collection of data was all new, none of the information being carried over from the earlier work. Very great care was used in checking all information, particularly with regard to the exact locations of accidents. The value of this care is demonstrated by the greater consistency of the results in their report.[2]

To sum up, the accident rate on two-lane tangents does not appear to be significantly affected by grade.

On three-lane tangents, as on the two-lane tangents, most of the travel was on roads of less than 3-percent grade. Large fluctuations are present in the accident rates, and no reliable relation is found between accident rate and the percentage of grade. The slopes for the types 1 and 3 rates lack statistical significance, as with the two-lane roads. The type 2 rate has a significant negative slope, with the rate declining as the grades become steeper; but the decline is meaningless because of a peculiar distribution of travel among the different States. The high rate for roads of less than 3-percent grade is caused entirely by the figures from one State, Oregon, which included no roads with higher grades. If Oregon data are excluded from the type 2 rate, the significant relation disappears.

Four-lane roads

On four-lane tangents without a median, it seems unlikely that the grade has any effect on the accident rate, even though the type 1 rate does have a significant upward slope of 0.69 ± 0.48 per percent of grade. (This means that the rate increases by about 0.69 for each 1-percent increase in the grade. There is a 95-percent chance that the true slope is between $0.69 - 0.48 = 0.21$ and $0.69 + 0.48 = 1.17$). The type 3 rate, which uses the very same accidents and vehicle-mileage, tends to become smaller as the grades increase but in a way that does not indicate a statistically significant trend. The type 2 rate is irregular but shows a slight tendency to increase as the grades increase.

This is an example of how each type of accident rate can point to a different conclusion. It appears that grades in the range used on main rural highways do not—when other factors are ignored—have any appreciable effect on the accident rate for four-lane undivided tangents, especially in view of the fact that the significant rate of increase for the type 1 rate is the result of a peculiar circumstance which distorts the true picture. Virginia, whose adjusted accident rate is much higher than that of any of the other States, contributes increasing proportions of the total travel as the grades

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[2] *Minnesota Rural Trunk Highway Accident, Access Point and Advertising Sign Study* (1951).

Table 8.—Accident rates on two-lane tangents, by width of shoulders

[rates are per million vehicle-miles]

Shoulder width, in feet	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
Curb	10	2.9	3	0.8	10	1.4
0–4.9	2,673	3.9	2,012	3.1	2,673	2.6
5–7.9	2,789	3.6	1,556	3.1	2,789	2.0
8–9.9	525	3.6	338	3.0	525	2.4
10 or more	476	4.1	350	3.3	476	2.8

Table 9.—Accident rates (type 3) on two-lane tangents, by volume of traffic, pavement width, and shoulder width

[rates are per million vehicle-miles]

Pavement width, in feet	Number of accidents and accident rate when shoulder width is—									
	Less than 5 feet		5 to 7.9 feet		8 to 9.9 feet		10 feet or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY										
16 or less	97	2.8	96	5.2	0	.....	1	3.3	194	3.6
18	679	2.0	871	2.1	82	2.0	58	3.8	1,690	2.1
20	991	2.7	1,027	1.6	223	1.8	66	1.6	2,307	2.0
21–22	193	2.4	25	1.4	65	2.5	117	2.9	400	2.4
23–24	67	2.5	167	1.5	16	2.1	9	2.7	259	1.7
25 or more	35	2.3	61	1.6	0	.....	55	2.6	151	2.1
Total	2,062	2.4	2,247	1.8	386	2.0	306	2.5	5,001	2.1
5,000–9,900 VEHICLES PER DAY										
16 or less	4	4.0	48	20.0	0	.....	0	....	52	15.3
18	14	1.7	73	3.5	12	2.0	2	2.5	101	2.8
20	469	3.5	237	3.0	104	7.9	71	2.9	881	3.5
21–22	49	3.0	29	2.7	23	11.5	5	1.7	106	3.3
23–24	13	9.3	27	7.3	0	......	0	.....	40	6.8
25 or more	33	3.9	88	2.9	0	...	92	4.4	213	3.6
Total	582	3.4	502	3.4	139	6.3	170	3.5	1,393	3.6
ALL VOLUMES										
16 or less	101	2.8	144	7.0	0	...	1	3.3	246	4.3
18	693	2.0	944	2.1	94	2.0	60	3.7	1,791	2.1
20	1,497	2.9	1,297	1.8	327	2.4	137	2.1	3,258	2.3
21–22	242	2.5	54	1.9	88	3.2	122	2.8	506	2.6
23–24	80	2.8	194	1.7	16	1.9	9	2.7	299	1.9
25 or more	68	2.8	150	2.2	0	......	147	3.5	365	2.7
Total	2,681	2.6	2,783	2.0	525	2.4	476	2.8	6,465	2.2

Table 10.—Accident rates on two-lane tangents, by frequency of curves

[rates are per million vehicle-miles]

Number of curves per mile	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
0–0.4	1,251	4.0	442	3.1	1,251	1.7
0.5–0.9	1,463	3.9	649	3.4	1,463	2.1
1.0–1.4	580	3.4	329	2.7	580	2.0
1.5–1.9	771	3.4	573	2.4	771	2.2
2.0–2.9	588	4.2	492	3.8	588	3.1
3.0–3.9	552	3.1	508	3.0	552	2.6
4.0–4.9	806	3.8	806	3.5	806	3.5
5.0–5.9	405	3.2	405	3.0	405	3.0
6.0–6.9	55	5.5	55	4.3	55	4.3
7 or more	0	.....	0	....	0	....

increase. This makes the type 1 rate appear to increase, even though there is no increase when Virginia is considered by itself or when all the other States are considered with only Virginia excluded (see further discussion of Virginia, p. 177).

For four-lane tangents having a median but no control of access, the accident rates are also inconclusive. None of the slopes is statistically significant, but there is some tendency for the rate to decline as the grade increases. On four-lane divided roads with controlled access, too, the grade has no particular effect on the accident rates.

In summary, on tangent highway sections there does not appear to be any relation between grade and accident rates. In these analyses the roads have been classified only by grade, so it remains possible that grade may have some effect on the accident rate when the appropriate other features are held constant.

Tangents: Effect of Volume

Two- and three-lane roads

Figure 3A shows how the accident rate on two-lane tangents varies with the average daily traffic when all other characteristics of the highway section are ignored. This is a typical example of the type of bar graph used in this report. Each bar conveys three distinct pieces of information. The height of the bar indicates the type 3 accident rate for the set of roads which the bar represents. The horizontal position of the bar indicates the average daily traffic volume on the roads represented. The width (thickness) of the bar indicates the number of vehicle-miles of travel on which the rate is based. No scale is shown for these widths, since it is only the relative widths that matter. The scales are different in each graph.

To be specific, the first bar in figure 3A shows that the type 3 rate is 1.3 for roads carrying from 0 to 900 vehicles per day. The next bar shows the rate to be 1.6 for roads carrying from 1,000 to 1,900 vehicles per day, and that this rate is based on about five times as much experience as the first rate. And so on.

The graph suggests a definite pattern, which in fact is the same for all three types of rates. (Information concerning all three types of rates is presented in table 6.) The accident rate increases steadily with increasing volume, reaching a maximum for roads carrying 8,000 to 9,000 vehicles per day. Heavier traffic reduces the accident rate somewhat, presumably because the extreme congestion at such high volumes makes it difficult for drivers to engage in passing maneuvers. The latter point is of small interest to the highway designer, who would hardly recommend two-lane construction for a road expected to carry as many as 9,000 vehicles per day.

In the range that is of principal interest the relation is simple and straightforward: higher traffic volumes mean higher accident rates.

There is a similar increase on three-lane tangents, as shown in figure 3B. The type 1 and type 3 rates both increase significantly as the traffic volume becomes larger. The information for the type 2 rate is too fragmentary to be of much value.

Four-lane roads

Figure 3C represents the condition on four-lane undivided tangents. All three types of accident rates have a pattern similar to that for the two-lane tangents: the rate goes up until a certain volume is reached, after which it drops down again. But the three types of rates do not have their maxima at the same traffic volume. The type 1 rate reaches its peak between 5,000 and 10,000 vehicles per day, while the type 2 and type 3 rates are highest for volumes between 15,000 and 20,000. The type 1 rate would have its peak in this same range if the Virginia figures were omitted. The Virginia data play a disturbing role in many of the type 1 rates. They have a low total-to-fatal ratio, with the consequently high adjustment factor of 4.59. Yet this adjustment factor seems excessive, for the adjusted accident rates are usually much higher for Virginia than for the other States. For example, on four-lane undivided tangents carrying between 5,000 and 9,900 vehicles per day, the adjusted accident rate for Virginia is 10.4 while it is only 3.7 for all the other States combined. Both rates are based on more than 200 accidents.

Figure 3D shows the same information for four-lane divided tangents without controlled access. The pattern is the same as before, with the accident rate going up as traffic volume increases. If there is any volume above which the accident rate begins to drop, it is beyond the range of these data, for which the maximum volume is 20,000 vehicles per day. The type 1 rate is higher in the 10,000 to 14,900 group than in the 15,000 to 19,900 group, but this is due to the peculiar effect of Virginia discussed previously.

The accident rates for four-lane divided roads with controlled access are shown in figure 3E. The rates appear to be somewhat lower for high volumes than for low volumes, but the appearance is misleading. The volumes under 5,000 come almost exclusively from Pennsylvania, while the volumes over 5,000 are all from Connecticut. The comparison is not so much between low volumes and high volumes as between Pennsylvania and Connecticut. Pennsylvania's accident rate is higher than Connecticut's, even though the average volumes are 4,000 and 15,000 respectively. Examination of the trend within each State shows that in Pennsylvania, where the volumes range from 3,000 to 5,000 vehicles per day, the accident rate increases steadily with increasing volume, but it is hard to draw conclusions from such a small range of volumes. In Connecticut the range is wide, and there is no significant trend. The evidence at hand does not indicate that traffic volume has any particular effect on the accident rate on four-lane divided tangents with controlled access.

Summary

The foregoing material is summarized in table 6. In most cases the average daily traffic has a considerable effect on the accident rate on tangent highway sections. The common pattern is for the accident rate to increase as the volume increases. At very high volumes the accident rate usually drops somewhat, probably because of congestion.

Table 11.—Accident rates (type 3) on two-lane tangents, by volume of traffic, frequency of curves, and length of tangent

[rates are per million vehicle-miles]

Number of curves per mile	Number of accidents and accident rate when length of tangent is									
	Less than 1 mile		1 to 1.9 miles		2 to 2.9 miles		3 miles or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY										
Less than 0.5	66	1.9	64	1.5	135	2.5	881	1.6	1,146	1.7
0.5-0.9	213	1.9	279	2.2	197	2.0	466	1.5	1,155	1.8
1-1.9	492	2.0	232	1.7	75	1.5	231	2.6	1,030	2.0
2-2.9	342	2.7	51	2.8	4	1.0	7	3.3	404	2.7
3 or more	1,071	2.9	95	2.4	26	2.8	26	4.9	1,218	2.9
Total	2,184	2.5	721	2.0	437	2.0	1,611	1.7	4,953	2.0
5,000-9,900 VEHICLES PER DAY										
Less than 0.5	16	8.4	0	....	0	......	68	4.0	84	4.4
0.5-0.9	88	4.7	17	3.5	119	4.9	63	3.8	287	4.5
1-1.9	190	2.9	33	1.4	0	......	45	3.8	268	2.6
2-2.9	128	4.1	51	5.3	0	......	0	......	179	4.3
3 or more	408	3.2	93	4.8	19	5.3	31	8.4	551	3.6
Total	830	3.4	194	3.4	138	4.8	207	4.2	1,369	3.6
ALL VOLUMES										
Less than 0.5	82	2.2	64	1.5	135	2.5	949	1.6	1,230	1.7
0.5-0.9	301	2.3	296	2.2	316	2.6	529	1.6	1,442	2.0
1-1.9	714	2.2	265	1.7	75	1.5	276	2.8	1,330	2.1
2-2.9	470	3.0	102	3.7	4	1.0	7	3.3	583	3.1
3 or more	1,488	2.9	188	3.7	73	4.1	57	6.3	1,806	3.1
Total	3,055	2.7	915	2.2	603	2.4	1,818	1.8	6,391	2.3

Table 12.—Accident rates (type 3) on two-lane tangents, by volume of traffic, frequency of intersections, and frequency of structures

[rates are per million vehicle-miles]

Number of intersections per mile	Number of accidents and accident rate when the number of structures per mile is—					
	Less than one		One or more		Total	
	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY						
Less than 0.5	357	1.8	19	1.9	376	1.8
0.5-0.9	798	1.9	47	1.3	845	1.8
1-1.9	2,425	1.9	280	2.2	2,705	1.9
2-2.9	675	2.8	28	7.6	703	2.9
3 or more	359	2.7	13	5.9	372	2.7
Total	4,614	2.0	387	2.2	5,001	2.1
5,000-9,900 VEHICLES PER DAY						
Less than 0.5	23	5.0	0	......	23	5.0
0.5-0.9	23	11.5	0	......	23	11.5
1-1.9	382	4.3	87	4.0	469	4.2
2-2.9	553	3.5	48	20.0	601	3.8
3 or more	278	2.5	0	......	278	2.5
Total	1,259	3.5	135	5.6	1,394	3.6
ALL VOLUMES						
Less than 0.5	380	1.9	19	1.9	399	1.9
0.5-0.9	821	1.9	47	1.3	868	1.9
1-1.9	2,835	2.1	367	2.5	3,202	2.1
2-2.9	1,262	3.1	76	12.5	1,338	3.2
3 or more	646	2.6	13	5.9	659	2.6
Total	5,944	2.2	522	2.6	6,466	2.3

As between the different types of roads at the same volumes, the conclusions depend on which type of accident rate is examined. Judged by the type 1 rate, the safest roads at volumes below 10,000 vehicles per day are the four-lane divided roads without controlled access, followed closely by the two-lane roads; the four-lane undivided roads are the worst in this volume range. Above 10,000 vehicles per day the four-lane divided roads with controlled access are far and away the safest, while the three-lane roads have much the highest accident rates.

The type 3 rates show little difference between road types for volumes under 5,000

Table 13.—Accident rates on two-lane tangents, by frequency of roadside establishments (including dwellings)

[*rates are per million vehicle-miles*]

Number of establishments per mile	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
0–0.9						
1.0–4.9	650	4.3	220	5.3	650	1.8
5.0–9.9	2,131	3.4	904	2.6	2,131	1.8
10.0–19.9	1,567	4.4	1,205	3.3	1,567	2.9
20.0–49.9	1,770	3.5	1,637	3.2	1,770	2.9
50 or more	356	4.0	293	2.9	356	3.1
	0	....	0	....	0	....

Table 14.—Accident rates on two-lane tangents, by frequency of sight-distance restrictions

[*rates are per million vehicle-miles*]

Number of restrictions per mile	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
0–0.9	3,472	3.7	1,833	2.8	3,472	2.0
1.0–1.9	1,061	4.3	588	4.0	1,061	2.5
2.0–2.9	891	4.1	811	3.4	891	3.1
3.0–3.9	684	3.3	661	3.0	684	3.0
4.0–4.9	354	3.1	354	2.9	354	3.0
5.0–5.9	12	2.7	12	2.7	12	2.7

vehicles per day, while the two-lane roads have the *highest* type 3 rate for volumes between 5,000 and 10,000. Above 10,000 the conclusion is the same as before: the three-lane roads are the most hazardous, while the controlled-access four-lane divided roads are the safest.

The type 2 rates are still different. Below 5,000 vehicles per day information is fragmentary. Between 5,000 and 10,000, the three-lane roads are the worst, while the four-lane divided roads without controlled access are the safest of those for which the samples are adequate. Above 10,000 vehicles per day the conclusion is the same as for the types 1 and 3 rates.

Two-Lane Tangents: Effects of Other Features

Since traffic volume has a pronounced effect on the accident rates, it is desirable to group the roadway sections by volume before studying the effects of other factors. Alternatively, the volume itself can be made one of the independent variables in a multiple regression analysis, a procedure which separates the effects of different factors. Both approaches have been used.

With grade and volume as independent variables, the multiple analysis corroborates the earlier conclusion that grade has no statistically significant effect on the accident rate, while volume does.

Pavement width, shoulder width, and traffic volume

If the two-lane tangents are classified solely according to their pavement width—irrespective of traffic volume, shoulder width, or other factors—the results are as shown in table 7. The evidence is confusing. It is not even clear whether 24-foot pavements are safer than 20-foot pavements; the type 3 rate suggests that they are, while the types 1 and 2 rates indicate that they are not. Neither is it definitely established that very narrow pavements have the highest accident rates. The types 1 and 3 rates indicate that they do, but the type 2 rate shows pavements of 16 feet as having a lower accident rate than those of 24 feet.

Similar information about the effect of shoulder width is presented in table 8. There is no indication that shoulder width, considered alone, has any bearing on the accident rates.

With the roads grouped according to traffic volume in 5,000 vehicle-per-day intervals, a multiple analysis has been made in each group, using pavement width and shoulder width as the independent variables. The complete table is too complicated for inclusion here, but a condensed version is given in table 9.

None of the effects is statistically significant. In neither the 0 to 4,900 volume group nor the 5,000 to 9,900 group is there a statistically significant effect on either the type 1 or the type 3 accident rate due to (1) pavement width, for constant shoulder width; (2) shoulder width, for constant pavement width; or (3) pavement width and shoulder width acting together. (For (1) and (2) the statistical tests are t-tests of the partial regression coefficients. For (3) they are F-tests of the multiple correlation coefficients. The two types of tests are equivalent when there is only one independent variable.) The material in this study indicates that neither pavement width nor shoulder width nor any combination of them has a determinable effect on the accident rates on two-lane tangents.

Frequency of curves

In the belief that driver behavior and accident experience might be affected by the frequencies of occurrence of curves, intersections, and other such features, the study routes have been divided into "frequency sections" averaging 10 to 15 miles in length. Every card contains all the frequency information for the frequency section to which it belongs.

Table 10 shows how the accident rates on two-lane tangents vary with the average number of curves per mile. As usual, the result depends on which figures are examined. The only rates with a significant trend are the type 3 rates, which go up as the curves become more frequent. The types 1 and 2 rates suggest an opposite con-

Table 15.—Accident rates (type 3) on two-lane tangents, by volume of traffic, percentage of commercial traffic, and percentage of night traffic

[*rates are per million vehicle-miles*]

Commercial traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is night traffic is—							
	0 to 19 percent		20 to 29 percent		30 to 39 percent		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY								
0–9.9	1	5.0	0	....	108	2.8	109	2.8
10–14.9	786	3.0	281	2.1	503	2.7	1,570	2.7
15–19.9	249	3.7	564	1.6	470	2.5	1,283	2.1
20–24.9	8	1.3	866	2.2	850	1.4	1,724	1.7
25 or more	0	....	221	1.7	94	1.3	315	1.6
Total	1,044	3.1	1,932	1.9	2,025	1.8	5,001	2.1
5,000–9,900 VEHICLES PER DAY								
0–9.9	0	....	0	....	68	2.5	68	2.5
10–14.9	303	6.4	183	4.2	194	2.8	680	4.2
15–19.9	204	6.8	0	....	235	2.7	439	3.8
20–24.9	0	....	72	3.5	111	2.3	183	2.7
25 or more	0	....	9	1.4	15	1.9	24	1.7
Total	507	6.5	264	3.8	623	2.6	1,394	3.6
ALL VOLUMES								
0–9.9	1	5.0	0	....	176	2.7	177	2.7
10–14.9	1,117	3.6	464	2.6	697	2.8	2,278	3.1
15–19.9	453	4.7	564	1.6	739	2.6	1,756	2.4
20–24.9	8	1.3	938	2.3	970	1.5	1,916	1.8
25 or more	0	...	230	1.7	109	1.4	339	1.6
Total	1,579	3.8	2,196	2.0	2,691	2.0	6,466	2.3

clusion, that the tangent sections interspersed with one or two curves per mile are safer than those in places where curves are quite rare. Either conclusion, once established, has a plausible explanation, but the figures are confusing. Even the simple fact that all three types of rates have their highest values for curve frequencies of six or more curves per mile is not so simple as it seems, for the State which supplied all these sections, Oregon, has a still higher accident rate for frequencies between 4.0 and 4.9.

Table 11 gives a multiple breakdown of the type 3 accident rates by curve frequency, tangent length, and traffic volume. The length is that of the entire tangent, even though it may be broken up into a number of shorter sections by the presence of minor intersections or structures. The multiple regression analysis corroborates the confusing conclusions from the analysis of curve frequency alone. In the lowest volume group, the effect of adding curves is to reduce the type 1 accident rate and to increase the type 3 rate. There is no significant effect at higher volumes.

The value of the multiple analysis becomes apparent when we examine the effect of tangent length on the type 3 accident rates for volumes under 5,000 vehicles per day. The totals for all curve frequencies combined indicate a high positive correlation between accident rate and tangent length. Even when the detailed breakdown is used, the simple correlation with tangent length is still statistically significant. Yet it falsifies the truth. For there is a large negative correlation between tangent length and curve frequency—i.e., long tangents have a strong tendency to be associated with low curve frequencies. To determine the effect of different tangent lengths on roads having the same curve frequency we must use the *partial* correlation coefficient of accident rate with tangent length. This coefficient is not statistically significant for any volume group.

Frequency of other features

A breakdown of accident rates according to structure frequency, intersection frequency, and traffic volume is given in table 12. There are no statistically significant effects of structure frequency or intersection frequency. However, there is some tendency for the type 3 rates to rise with increasing intersection frequency when the traffic volume is low, and to fall with increasing intersection frequency when the traffic volume is high.

Table 13 shows the effect on the accident rates of the frequency of roadside establishments (including dwellings). The results are perplexing. Only the type 3 rate comes anywhere near showing a significant trend; these figures suggest that adding roadside establishments makes a road more hazardous. The type 2 rate, on the other hand, seems to indicate that the only roads that are particularly bad are those having less

Table 16.—Accident rates on curves, by degree of curvature and roadway type

[rates are per million vehicle-miles]

Curvature, in degrees	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
0–2.9	504	2.6	11	5.6	98	4.9	95	2.4	180	2.4
3–5.9	596	3.6	11	9.8	90	8.4	65	4.2	162	3.4
6–9.9	338	3.6	6	14.1	16	7.9	5	11.9	38	5.6
10 or more	354	4.8	11	28.0	3	5.8	12	30.6	0	....
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
0–2.9	340	1.8	0	......	43	1.9	33	0.7	180	1.6
3–5.9	447	2.5	0	....	33	2.1	52	2.7	162	2.3
6–9.9	287	2.9	0	....	10	2.9	1	1.2	38	4.5
10 or more	281	3.4	1	10.0	0	....	0	....	0	....
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
0–2.9	504	1.6	11	1.7	98	1.9	95	1.8	180	1.6
3–5.9	596	2.5	11	2.8	90	2.6	65	2.4	162	2.3
6–9.9	338	2.8	6	3.5	16	3.3	5	3.1	38	4.5
10 or more	354	3.5	11	7.3	3	1.2	12	6.7	0	.

Table 17.—Accident rates on tangents and curves,[1] by roadway type

[rates are per million vehicle-miles]

Location	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
Tangents	6,474	3.7	227	6.1	1,348	6.4	982	4.6	774	2.2
Curves	1,794	3.3	39	10.2	210	6.5	177	3.8	380	2.9
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
Tangents	4,259	3.1	115	5.3	757	3.3	829	2.9	774	1.7
Curves	1,355	2.5	1	2.5	86	1.9	86	1.3	380	2.0
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
Tangents	6,474	2.3	227	2.5	1,348	2.7	982	2.9	774	1.7
Curves	1,794	2.3	39	2.8	210	2.2	177	2.1	380	2.0

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[1] All volumes, grades, curvatures, etc.

Table 18.—Accident rates on curves, by volume of traffic and roadway type

[rates are per million vehicle-miles]

Average daily traffic	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
0–4,900	1,387	3.5	21	9.1	25	5.7	1	1.1	140	3.8
5,000–9,900	403	3.0	18	11.7	96	7.6	43	4.4	0	..
10,000–14,900	4	0.6	0		69	3.4	117	4.1	45	1.8
15,000 or more	0	.	0		20	1.9	27	6.5	63	1.4
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
0–4,900	957	2.5	1	2.5	2	0.6	0	.....	140	1.9
5,000–9,000	394	2.7	0	.	20	1.5	18	0.7	0	..
10,000–14,900	4	0.6	0		34	2.0	111	2.9	45	1.8
15,000 or more	0		0		20	1.8	27	5.9	63	1.3
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
0–4,900	1,387	2.3	21	2.6	25	1.7	1	0.3	140	1.9
5,000–9,900	403	2.7	18	3.1	96	2.3	43	1.4	0	..
10,000–14,900	4	0.6	0	....	69	2.4	117	2.9	45	1.8
15,000 or more	0	....	0		20	1.8	27	5.9	63	1.3

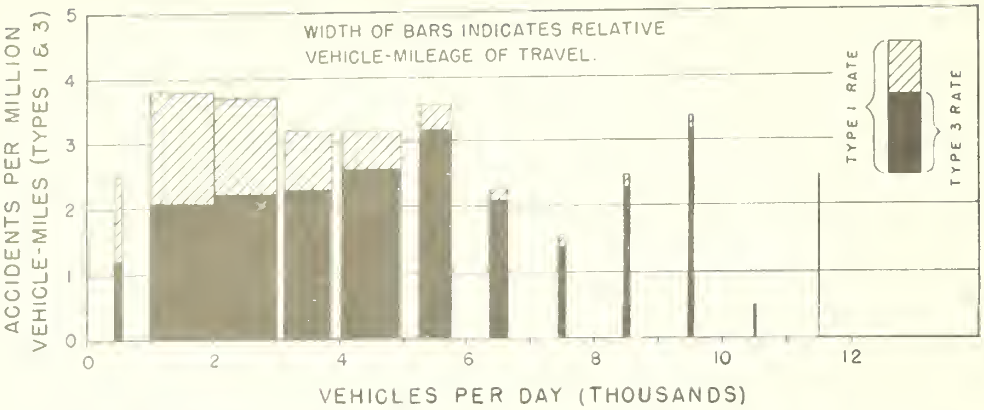

Figure 5.—Accident rates on two-lane curves (all degrees), by volume of traffic.

than one establishment per mile. The type 1 rate is quite irregular. The effect on the accident rates of the frequency of roadside establishments has been studied in more detail by the States of Minnesota and Michigan.[1] The Minnesota study is cited in footnote 2 (p. 175).

In studying the frequency of sight-distance restrictions a restriction has been defined as a stretch of road where the sight distance is less than 600 feet in flat or rolling terrain, or less than 400 feet in mountainous terrain. The relation of accident rates to the frequency of sight restrictions is shown in table 14. The type 3 rates are the most meaningful. Their slope is statistically significant, with the accident rate rising as the restriction frequency increases from zero up to about three restrictions per mile. The types 1 and 2 rates have maxima when there are between one and two restrictions per mile; they drop steadily as the frequency of restrictions increases above this number.

Commercial and night traffic

Table 15 presents a three-way breakdown of the type 3 accident rates by traffic volume, the percentage that is commercial traffic, and the percentage of the traffic that flows after dark. The multiple analysis indicates that the accident rate is reduced as the percentage of night traffic increases, when the percentage of commercial traffic remains the same; and that the accident rate also falls off with increasing commercial traffic when the night traffic is held fixed. Both these effects were unexpected.

In summary, of all the characteristics studied for their effects on the accident rate on two-lane tangents, traffic volume is

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[1] Michigan study, *Accident Experience in Relation to Road and Roadside Features* (1952).

Table 19.—State-by-State accident rates (type 1) on two-lane curves, by volume of traffic

[*rates are per million vehicle-miles*]

State	Number of accidents, amount of travel, and accident rate when traffic volume is— 0 to 4,900 vehicles per day: Number of accidents (adjusted)	Travel, in million vehicle-miles	Accident rate	5,000 to 9,900 vehicles per day: Number of accidents (adjusted)	Travel, in million vehicle-miles	Accident rate
Colorado	238	71.2	3.3	12	1.3	9.2
Connecticut (1941)	269	95.1	2.8	220	80.9	2.7
Connecticut (1946)	282	112.0	2.5	69	29.2	2.4
Georgia	106	13.7	7.7	0	0	....
Iowa	220	40.5	5.4	0	1.4	....
Louisiana	56	27.4	2.0	2	0.2	10.0
Minnesota	41	33.8	1.2	0	0	....
Nebraska	201	54.7	3.7	0	0.4	....
New Mexico	50	5.0	10.0	0	0	....
Oregon	108	46.7	2.3	17	8.7	2.0
Utah	206	31.6	6.5	7	1.9	3.7
Virginia	23	3.0	7.7	0	0	....
Washington	294	63.2	4.7	120	27.0	4.4
Wisconsin	11	10.7	1.0	0	0	....
Wyoming	14	4.4	3.2	0	0	....
Total	2,119	613.0	3.5	447	151.0	3.0

Table 20.—Accident rates on two-lane curves, by volume of traffic and degree of curvature

[*rates are per million vehicle-miles*]

Curvature, in degrees	Number of accidents and accident rate when traffic volume is— 0 to 4,900 vehicles per day: Number	Rate	5,000 vehicles per day or more: Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)				
0–2.9	395	2.7	111	2.1
3.0–5.9	423	3.7	173	3.4
6.0 or more	569	4.1	123	3.1
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)				
0–2.9	231	1.8	109	2.0
3.0–5.9	278	2.3	169	3.1
6.0 or more	448	3.2	120	2.9
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)				
0–2.9	395	1.6	111	1.9
3.0–5.9	423	2.3	173	3.1
6.0 or more	569	3.2	123	2.8

Table 21.—Accident rates (type 3) on two-lane curves, by volume of traffic, degree of curvature, and grade

[*rates are per million vehicle-miles*]

Curvature, in degrees	Number of accidents and accident rate when grade is— Less than 3 percent: Number	Rate	3 percent or more: Number	Rate	Total: Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY						
0–2.9	317	1.4	78	2.0	395	1.6
3–5.9	317	2.3	106	2.4	423	2.3
6–9.9	194	3.0	69	2.3	263	2.8
10 or more	155	3.4	150	3.8	305	3.6
Total	983	2.1	403	2.7	1,386	2.3
5,000–9,900 VEHICLES PER DAY						
0–2.9	86	1.9	22	2.9	108	2.0
3–5.9	117	2.8	55	4.1	172	3.2
6–9.9	51	2.6	22	3.1	73	2.7
10 or more	27	2.5	22	3.9	49	3.0
Total	281	2.4	121	3.6	402	2.7
ALL VOLUMES						
0–2.9	405	1.6	100	2.2	505	1.6
3–5.9	434	2.4	161	2.8	595	2.5
6–9.9	245	2.9	93	2.5	338	2.8
10 or more	182	3.2	172	3.8	354	3.5
Total	1,266	2.2	526	2.8	1,792	2.3

the only one whose effect is entirely clear. The effects of the frequencies of curves, intersections, roadside establishments, and sight restrictions are all uncertain, while grade, pavement width, shoulder width, tangent length, and frequency of structures do not have any independent effects on the accident rates. The effects of commercial traffic and night traffic are inconclusive.

Curves: Effects of Curvature and Volume

Degree of curvature

The type 3 accident rates on two-lane curves, by degree of curvature, are presented in figure 4A (p. 174), and all three types of accident rates are given in table 16. The relation is clearcut: the sharper the curve, the higher the accident rate. As a matter of fact, the slopes are highly significant for all three types of rate. In accident-rate units per degree, the slopes are 0.19 ± 0.07 for the type 1 rate, 0.12 ± 0.05 for the type 2, and 0.16 ± 0.05 for the type 3. Thus, whichever type of accident rate is used, the number of accidents per million vehicle-miles increases by about 0.15 for each additional degree of curvature.

For three-lane roads, too, there seems to be a steady increase in hazard with increasing curvature, though the data are somewhat sparse. Figure 4B shows the rates based on a total of 39 accidents.

Figure 4C gives the corresponding information for four-lane undivided curves. The type 3 rate has the same upward trend as on the two- and three-lane roads, although the slope is not statistically significant; the types 1 and 2 rates are more irregular.

On four-lane divided roads the accident rate also increases as the curves become sharper. The rates are presented in figure 4D.

Figure 4E shows the relation of curvature to accident rate for four-lane divided roads with controlled access. As before, the trend is statistically significant, with the accident rates increasing by about 0.4 for each additional degree of curvature.

In summary, there is a direct relation between curvature and accident rate on all types of highways. Sharp curves have high accident rates, gradual curves have low accident rates, in-between curves have in-between accident rates.

Among different types of roads with the same degree of curvature, the data do not indicate any consistent relation.

Table 17 compares tangents with curves on each type of roadway. (These rates are for all the tangent and curve sections included in the study, irrespective of other factors.) There is no clear superiority one way or the other. The type 3 rates, which are the most consistent, indicate that tangents and curves are equally safe on two-lane roads. Tangents are a little safer than curves on three-lane and controlled-access four-lane divided roads, while they are somewhat more hazardous on the four-lane roads lacking control of access. None of these differences is large enough to warrant any strong conclusions.

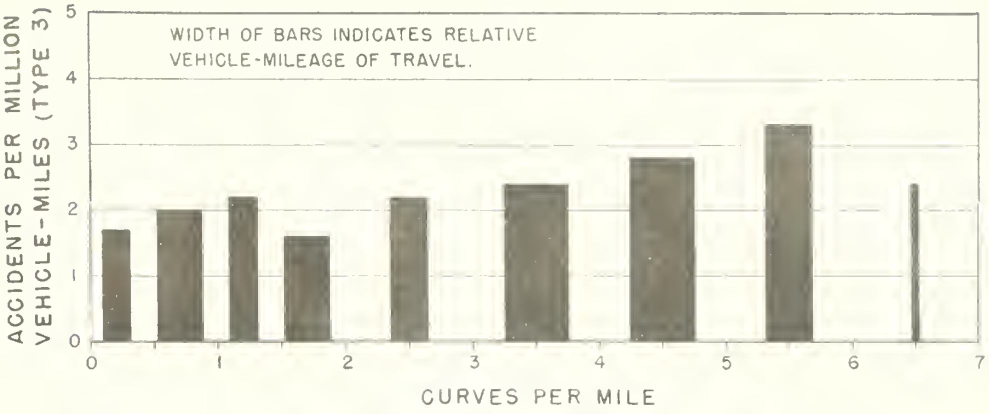

Figure 6.—Accident rates on two-lane curves (all degrees), by frequency of curves.

Effect of traffic volume

Table 18 and figure 5 show how the accident rate on two-lane curves varies with the average daily volume of traffic. The type 1 rate has a statistically significant tendency to become *smaller* as the traffic increases. The types 2 and 3 rates do not vary significantly but have a slight tendency to increase with increasing traffic.

The decline shown by the type 1 rate was unexpected, but the conclusion which it suggests is almost certainly correct. Table 19 gives a State-by-State breakdown of these rates. This table shows that every State having more than six accidents (actual, not adjusted) at volumes over 5,000 vehicles per day has a lower accident rate for these volumes than for volumes below 5,000. It has also been proved by multiple regression analysis that this decline is not a hidden effect of the degree of curvature.

Why should the accident rate decline with increasing traffic on two-lane curves, when it goes the opposite way on two-lane tangents? Analysis of the types of collisions at different volumes indicates that the distribution of accident types is practically identical for all volume groups. So one can only speculate. Perhaps the extra alertness required for driving on narrow curved roads in heavy traffic is what pulls the accident rate down. A similar decline in accident rate with increasing traffic volume is found at two-lane intersections, where the same alertness factor may be involved. It is not found at curves or intersections on wider roads.

For three-lane curves the data are pretty meager (see table 18), but such evidence as there is points to a positive correlation between accident rate and traffic volume.

On four-lane undivided curves each type of rate increases to a maximum and then falls off gradually as the traffic volume increases. The type 1 rate has its maximum between 5,000 and 10,000 vehicles per day, the type 2 has its maximum between 15,000 and 20,000, the type 3 has its maximum between 10,000 and 15,000. This is similar to the pattern for four-lane undivided tangents.

Table 22.—Accident rates on two-lane curves, by degree of curvature and frequency of curves

[*rates are per million vehicle-miles*]

Number of curves per mile	Number of accidents and accident rate when curvature is—							
	0 to 2.9 degrees		3 to 5.9 degrees		6 to 9.9 degrees		10 degrees or more	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)								
0–0.9	128	3.0	110	5.4	13	4.2	31	8.9
1.0–2.9	178	2.3	163	3.7	96	4.5	53	4.2
3.0–4.9	125	2.1	223	2.9	170	3.3	139	4.3
5.0–6.9	75	3.3	100	3.2	59	2.8	130	4.6
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)								
0–0.9	42	1.6	47	3.2	2	1.1	4	1.4
1.0–2.9	105	1.4	97	2.1	65	2.9	30	2.6
3.0–4.9	118	2.0	203	2.5	161	3.2	117	3.3
5.0–6.9	75	3.1	100	2.9	59	2.6	130	3.9
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)								
0–0.9	128	1.4	110	2.7	13	2.0	31	4.3
1.0–2.9	178	1.4	163	2.1	96	2.9	53	2.6
3.0–4.9	125	1.9	223	2.5	170	2.9	139	3.4
5.0–6.9	75	3.1	100	2.9	59	2.6	130	3.9

Table 23.—Accident rates (type 3) on two-lane curves, by volume of traffic, frequency of curves, and frequency of sight-distance restrictions

[rates are per million vehicle-miles]

Number of curves per mile	Number of accidents and accident rate when number of restrictions per mile is—									
	Less than 1		1 to 1.9		2 to 2.9		3 or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4.900 VEHICLES PER DAY										
Less than 0.5	76	1.7	7	1.9	0	....	3	7.5	86	1.7
0.5–0.9	138	1.8	33	3.2	3	0.9	0	....	174	1.9
1–1.9	182	1.8	40	1.7	3	0.9	0	....	225	1.7
2–2.9	38	1.9	98	2.4	16	1.7	0	......	152	2.1
3 or more	9	0.6	41	3.8	181	2.7	518	2.9	749	2.8
Total	443	1.7	219	2.4	203	2.5	521	2.9	1,386	2.3
5,000–9,900 VEHICLES PER DAY										
Less than 0.5	1	1.7	0	....	0	....	0	....	1	1.7
0.5–0.9	22	3.5	0	....	0	....	0	....	22	3.3
1–1.9	72	2.1	3	3.8	0	....	0	....	75	2.1
2–2.9	0	.....	36	2.4	0	....	0	....	36	2.4
3 or more	0	.....	4	1.1	131	2.7	133	3.2	268	2.9
Total	95	2.3	43	2.2	131	2.7	133	3.2	402	2.7
ALL VOLUMES										
Less than 0.5	77	1.7	7	2.2	0	....	3	7.5	87	1.7
0.5–0.9	160	1.9	33	3.0	3	0.9	0	....	196	2.0
1–1.9	256	1.8	43	1.7	3	0.9	0	....	302	1.8
2–2.9	38	1.9	134	2.4	16	1.7	0	....	188	2.2
3 or more	9	0.6	45	3.1	314	2.7	651	2.9	1,019	2.8
Total	540	1.8	262	2.4	336	2.5	654	2.9	1,792	2.3

The accident rates on four-lane divided curves show a persistent increase with traffic volume. The information is on the skimpy side, but the trend seems definite. The types 2 and 3 rates have statistically significant slopes, with the accident rate increasing by about 0.3 for each additional 1,000 vehicles per day.

For the four-lane divided curves with controlled access, the situation is similar to that of the four-lane divided tangents with controlled access. The apparent decline in the accident rate with increasing volume is mainly due to the difference between Pennsylvania and Connecticut. As in the earlier case, the high volumes are all from Connecticut, while practically all of the traffic below 5,000 vehicles per day is from Pennsylvania.

In summary, on all but the two-lane roads, the accident rate on curves varies with volume in much the same way as on tangents. The general tendency is for higher-than-average volumes to cause higher-than-average accident rates, with some decline in the accident rate at extremely high volumes.

The two-lane curves are different. They show a negative correlation between accident rate and traffic volume throughout the volume range. This is thought to be due to the greater care with which people drive under obviously dangerous conditions.

Two-Lane Curves: Effects of Other Features

Degree, grade, and volume

There are two basically different ways in which curvature and volume together might affect the accident rates. Even if the effects were really independent, there might be intercorrelation between the two factors—i.e., a tendency for the roads having higher-than-average curvature to have either higher-than-average or lower-than-average volume—so that an effect of curvature might appear to be an effect of volume, or vice versa. Multiple regression analysis separates the effects and assigns each to its proper cause.

Or there could be interaction between the two factors—the kind of situation in which the effect of volume at low degrees of curvature is different from its effect at high degrees, and the effect of curvature at low volumes is different from that at high volumes.

Both of these possibilities were investigated. The intercorrelation between degree of curvature and volume is negligible, and the partial correlations between the type 1 accident rate and each of the two factors are both statistically significant, with the same signs as the simple correlations. In other words, for roads of the same curvature the average effect of increased volume is to reduce the accident rate by a significant amount, and for roads carrying the same volume the average effect of increased curvature is to increase the accident rate by a significant amount.

The interaction between curvature and volume can be tested by making a two-way classification of the highway sections by both curvature and volume. This is done in table 20. The type 1 rate shows no particular interaction, since it drops with increasing volume in each curvature group. The types 2 and 3 rates do show interaction, and it is exactly the sort one would expect. At low degrees of curvature—i.e., on the curves which are most like tangents—the hazard increases with volume, just as on the tangent sections. On curves sharper than 6

Table 24.—Accident rates (type 3) on two-lane curves, by volume of traffic, pavement width, and shoulder width

[rates are per million vehicle-miles]

Pavement width, in feet	Number of accidents and accident rate when shoulder width is—									
	Less than 5 feet		5 to 7.9 feet		8 to 9.9 feet		10 feet or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY										
16 or less	28	1.5	5	1.1	0	....	0	....	33	1.4
18	257	2.7	153	2.0	4	1.3	1	0.7	415	2.4
20	473	3.0	219	1.9	40	1.4	33	2.2	765	2.4
21–22	58	2.3	5	0.7	2	1.7	26	2.3	91	2.0
23–24	22	2.5	31	1.4	1	0.3	0	....	54	1.6
25 or more	15	1.3	5	1.2	0	......	9	4.2	29	1.6
Total	853	2.7	418	1.8	47	1.3	69	2.3	1,387	2.3
5,000–9,900 VEHICLES PER DAY										
16 or less	1	0.9	1	0.7	0	......	0	....	2	0.8
18	7	3.5	12	1.2	5	5.0	0	....	24	1.8
20	230	3.2	84	2.4	3	1.2	10	2.0	327	2.9
21–22	6	2.7	13	2.3	0	....	3	3.3	22	2.5
23–24	0	....	2	.6	1	3.3	0	....	3	.8
25 or more	9	4.7	8	2.0	0	....	7	2.8	24	2.9
Total	253	3.2	120	2.0	9	2.4	20	2.4	402	2.7
ALL VOLUMES										
16 or less	29	1.5	6	1.0	0	....	0	....	35	1.4
18	264	2.7	165	1.9	9	2.2	1	0.7	439	2.3
20	705	3.1	305	2.0	43	1.4	43	2.1	1,096	2.5
21–22	64	2.3	18	1.4	2	1.7	29	2.4	113	2.1
23–24	22	2.5	33	1.3	2	0.6	0	....	57	1.5
25 or more	24	1.7	13	1.6	0	....	16	3.6	53	2.0
Total	1,108	2.8	540	1.8	56	1.4	89	2.3	1,793	2.3

degrees it is the other way around; here the accident rate is lower when there is more traffic volume.

Another way of looking at this interaction is in terms of the effect of changing the curvature at different fixed volume levels. This effect is the same for all three types of rates. At volumes below 5,000 vehicles per day, the accident rate rises steadily with increasing curvature. At volumes over 5,000 the accident rate is lower on sharp curves than on moderate curves.

These facts strengthen the belief that traffic volume affects the accident rate differently on two-lane curves from the way it does on two-lane tangents.

Table 21 gives the type 3 accident rates by degree of curvature and percentage of grade for two-lane curves carrying various ranges of traffic volume. At low volumes the accident rate goes up with increasing curvature, while the effect of grade is not statistically significant. At higher volumes it is not the curvature but the grade that matters; steeper grades make the accident rates larger. This peculiar pattern occurs with both the type 1 and the type 3 rates (the type 2 rates were not computed). In the 0 to 4,900 volume group the partial correlation with curvature is the only significant one, while in the 5,000 to 9,900 volume group the partial correlation with grade is the only significant one.

Curve frequency and other items

Figure 6 shows the accident rates on two-lane curves (of all degrees) as a function of curve frequency. The type 1 rate is highest when curves are very rare, suggesting that a curve is most hazardous when it is unexpected. The types 2 and 3 accident rates have their high values when there are more than five curves per mile.

Table 22 separates the figures into several ranges of curvature. At first glance the most prominent feature of this table is that the types 1 and 3 accident rates have their greatest values for the highest curvature and the lowest frequency. But closer study of the supporting facts shows that this is due to five curves in Iowa which contribute 11 out of the 31 accidents in this group. The other 20 accidents in the group occurred at rates similar to those for other curvatures and frequencies.

The table does not suggest any general conclusions about the effect of curve frequency on the accident rates. The degree of curvature, as before, is positively correlated with the accident rates.

Table 23 gives the accident rates on two-lane curves by curve frequency and sight-restriction frequency for various volume groups. There is a high intercorrelation between the two frequencies, so that while high values of both frequencies cause higher accident rates than low values of both frequencies, it is impossible to tell which of the two frequencies is responsible. (In statistician's language, the multiple correlation is statistically significant, but the partial correlations are not.)

Table 25.—Accident rates (type 3) on two-lane curves, by volume of traffic, percentage of commercial traffic, and percentage of night traffic

[rates are per million vehicle-miles]

Commercial traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is night traffic is—							
	0 to 19 percent		20 to 29 percent		30 to 39 percent		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY								
0–9.9	0	....	0	....	83	3.0	83	3.0
10–14.9	297	3.1	79	2.3	188	2.6	564	2.8
15–19.9	42	2.9	138	1.9	198	2.5	378	2.2
20–24.9	1	1.0	105	1.5	164	1.6	270	1.5
25 or more	0	....	71	2.9	21	2.0	92	2.6
Total	340	3.0	393	1.9	654	2.2	1,387	2.3
5,000–9,900 VEHICLES PER DAY								
0–9.9	0	....	0	....	38	3.0	38	3.0
10–14.9	88	3.7	11	1.5	92	2.4	191	2.7
15–19.9	18	3.5	0	....	64	2.1	82	2.1
20–24.9	0	....	3	2.5	83	2.9	86	2.9
25 or more	0	....	5	2.9	0	....	5	2.9
Total	106	3.6	19	1.8	277	2.5	402	2.7
ALL VOLUMES								
0–9.9	0	....	0	....	121	3.0	121	3.0
10–14.9	385	3.2	90	2.2	280	2.5	755	2.7
15–19.9	60	3.1	138	1.8	264	2.3	462	2.2
20–24.9	1	1.0	108	1.5	249	1.9	358	1.7
25 or more	0	....	76	2.9	21	2.0	97	2.8
Total	446	3.1	412	1.9	935	2.3	1,793	2.3

Table 26.—Accident rates at intersections at grade[1] on two- and three-lane roads, by total volume of traffic and percentage of cross traffic

[rates are per 10 million vehicles]

Cross traffic (percent of total)	Accidents on two-lane roads						Accidents on three-lane roads					
	0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day		0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)												
0–9	678	3.6	229	2.2	9	0.9	17	4.1	25	9.6	24	34.4
10–19	116	11.3	56	6.3	0	....	0	....	0	....	0	....
20 or more	162	9.2	118	8.7	3	1.9	0	....	30	40.8	0	....
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)												
0–9	363	2.0	213	1.8	9	0.8	2	3.3	6	2.1	24	26.7
10–19	63	7.1	54	5.5	0	....	0	....	0	....	0	....
20 or more	118	6.9	117	8.1	3	1.9	0	....	30	25.0	0	....
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)												
0–9	678	2.0	229	1.8	9	0.8	1	1.7	25	3.5	24	26.7
10–19	116	6.0	56	5.3	0	....	0	....	0	....	0	....
20 or more	162	6.5	118	7.5	3	1.9	0	....	30	25.0	0	....

[1] Excluding rotary intersections.

Table 27.—Accident rates at intersections at grade[1] on four-lane roads, by total volume of traffic and percentage of cross traffic

[rates are per 10 million vehicles]

Cross traffic (percent of total)	Accidents on undivided roads						Accidents on divided roads[2]					
	0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day		0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)												
0–9	33	8.8	184	9.9	302	4.6	15	3.6	131	5.4	309	7.8
10–19	15	22.9	22	48.7	123	32.3	7	17.5	25	10.8	130	28.3
20 or more	0	. . .	34	56.7	236	42.2	0	. . . . . .	13	13.6	97	22.4
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)												
0–9	13	4.6	62	3.8	255	3.1	15	3.3	91	3.4	238	4.5
10–19	14	28.0	8	8.0	123	28.6	7	17.5	25	11.4	130	24.5
20 or more	0	. . . .	15	21.4	227	36.0	0	. . . .	13	11.8	97	19.4
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)												
0–9	33	3.4	184	3.5	302	3.0	15	3.2	131	3.2	309	4.2
10–19	15	21.4	22	14.7	123	28.6	7	17.5	25	10.0	130	24.5
20 or more	0	. . . .	34	18.9	236	35.2	0	. . . .	13	11.8	97	19.4

[1] Excluding rotary intersections. [2] Excluding those with controlled access.

Since the preceding section seemed to indicate that curve frequency did not have any consistent effect, it can only be concluded that the effect of curve frequency is not very clear.

A breakdown of the effects of pavement width, shoulder width, and traffic volume is given in table 24. On two-lane curves the effect of pavement width is too irregular to be statistically significant, but 24-foot wide sections are consistently safer than 20-foot sections. The partial correlation with shoulder width is significant; the accident rate goes down by about 0.15, on the average, for each additional foot of shoulder. These results differ from the situation on two-lane tangents, where neither pavement width nor shoulder width has a consistent effect on the accident rate.

Table 25 shows the effects, on two-lane curves, of commercial traffic and night traffic. Neither effect is statistically significant.

In summary, the effect of volume in reducing the accident rate on two-lane curves seems to be stronger for sharp curves than for flat curves. This is logical, for one would expect flat curves to be intermediate, in their accident potential, between sharp curves and tangents.

The frequency of curves does not appear to have any consistent effect on the accident rate, even when the curves are subdivided by degree of curvature. The frequency of sight restrictions has a similarly uncertain effect.

Wide shoulders definitely help to reduce the accident rate on two-lane curves, and 24-foot pavements are consistently safer than 20-foot pavements. This is in contrast to the situation on two-lane tangents, where shoulder width has no particular effect.

The effect of grade is peculiar. At low traffic volumes there is no particular effect, but on roads carrying more than 5,000 ve-

Table 28—Accident rates (type 3) at intersections at grade[1] on two-lane roads, by total volume of traffic, percentage of cross traffic, type of intersection, and percentage of night traffic

[rates are per 10 million vehicles]

Night traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is cross traffic is—											
	0 to 9 percent						10 percent or more					
	Three-way intersection		Four-way intersection		Total		Three-way intersection		Four-way intersection		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY												
10–19	83	2.9	48	5.0	131	3.4	18	15.0	30	23.1	48	19.2
20–29	143	2.0	185	3.9	328	2.8	58	5.4	67	7.1	125	6.2
30–39	112	0.9	107	1.8	219	1.2	53	4.5	52	5.3	105	4.9
Total	338	1.5	340	2.9	678	2.0	129	5.4	149	7.2	278	6.3
5,000–9,900 VEHICLES PER DAY												
10–19	33	2.8	39	5.2	72	3.7	43	13.4	18	13.8	61	13.6
20–29	35	3.3	19	5.0	54	3.8	9	11.2	23	14.4	32	13.3
30–39	69	0.9	31	2.2	100	1.1	58	4.3	37	6.1	95	4.8
Total	137	1.3	89	3.5	226	1.8	110	6.3	78	8.7	188	7.1
ALL VOLUMES												
10–19	118	2.8	89	5.1	207	3.5	61	13.9	48	18.5	109	15.6
20–29	178	2.2	204	4.0	382	2.9	67	5.8	90	8.1	157	6.9
30–39	186	0.9	138	1.9	324	1.1	112	4.3	89	5.4	201	4.7
Total	482	1.4	431	3.0	913	1.9	240	5.7	227	7.5	467	6.5

[1] Excluding rotary intersections, interchanges at grade separations, and intersections with more than four approaches.

hicles per day the accident rates are higher on roads with steep grades.

Finally, the percentages of commercial and night traffic have no recognizable effect on the accident rates on two-lane curves.

Intersections

The accident rates at intersections are computed somewhat differently than on tangents and curves, since the length of an intersection is not particularly relevant to its accident potential. The base which has been used instead of vehicle-mileage is the total number of vehicles using the intersection. To keep the numbers at a manageable size, the accident rates have been expressed in terms of the number of accidents per 10 million vehicles. The only intersections analyzed have been intersections at grade, excluding rotaries, and the number of lanes at the intersections are those along the roads included in the study, irrespective of the number of lanes on the side or cross road. By volume is meant the total number of vehicles entering the intersection from all approaches; the percentage of cross traffic is the percentage of this total volume which enters the intersection from roads other than the study route.

Table 26 presents the accident rates for two-lane roads at these intersections, for various traffic volumes and percentages of cross traffic. The percentage of cross traffic is of crucial importance. At every volume level where there is an adequate sample, the accident rate is more than twice as high when the cross traffic exceeds 10 percent of the total as when the cross traffic is less than 10 percent. Additional increases in cross traffic to 20 percent or more do not cause appreciable further increases in the accident rates.

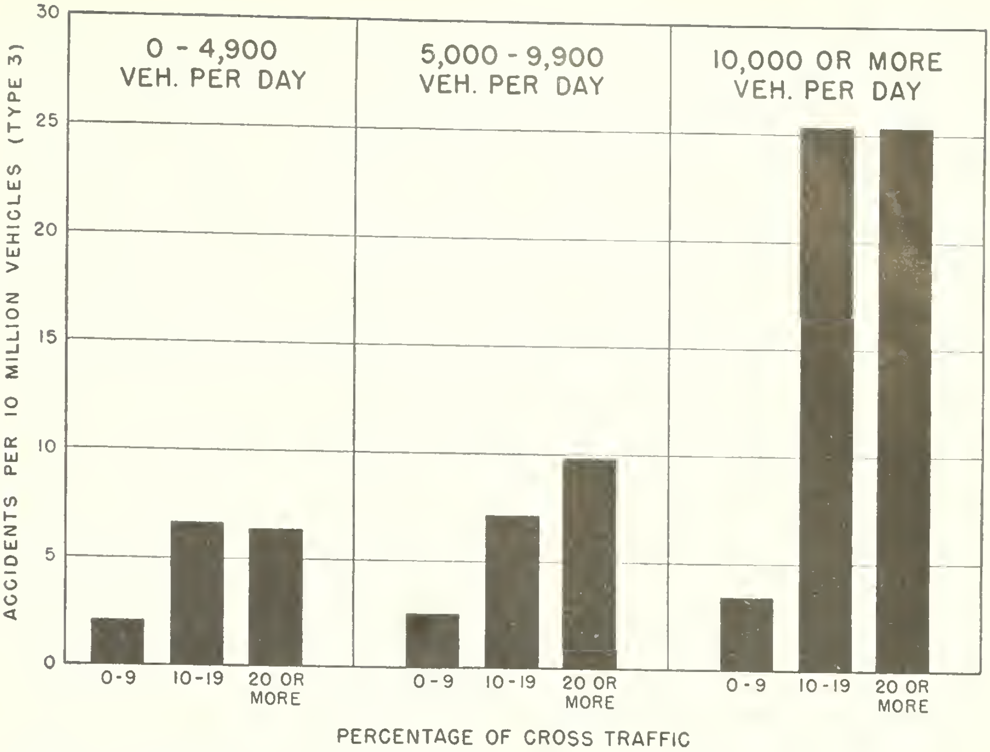

Figure 7.—Accident rates at intersections (all roadway types), by total volume and percentage of cross traffic.

Table 29.—Accident rates (type 3) at intersections at grade[1] on two-lane roads by total volume of traffic, percentage of cross traffic, and percentage of commercial traffic

[rates are per 10 million vehicles]

Commercial traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is cross traffic is—			
	0 to 9 percent		10 percent or more	
	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY				
5–9.9	11	0.6	8	3.3
10–14.9	208	2.3	88	6.4
15–19.9	174	1.9	121	6.3
20–24.9	215	1.8	57	6.0
25 or more	80	3.1	16	9.4
Total	688	2.0	290	6.3
5,000–9,900 VEHICLES PER DAY				
5–9.9	6	0.5	23	6.6
10–14.9	128	2.7	83	7.5
15–19.9	65	1.7	64	9.1
20–24.9	21	.8	39	4.6
25 or more	8	1.9	2	2.9
Total	228	1.7	211	6.9
ALL VOLUMES				
5–9.9	17	0.5	31	5.3
10–14.9	340	2.4	172	6.7
15–19.9	244	1.8	192	7.0
20–24.9	236	1.6	96	5.2
25 or more	88	2.9	18	7.5
Total	925	1.9	509	6.4

[1] Excluding rotary intersections.

Table 30.—Accident rates at structures on two-lane roads with approach pavements less than 30 feet wide, by relative width of structure roadway and adjoining pavement

[rates are per 10 million vehicles]

Relative width	Number of accidents and accident rate at—			
	Bridges and overpasses		Underpasses	
	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)				
Structure narrower by more than 1 foot	21	9.2	0	......
Structure from 1 foot narrower to 1 foot wider	56	5.8	2	2.9
Structure wider by:				
1.1–3.0 ft	81	7.7	6	6.4
3.1–5.0 ft	87	5.2	2	7.5
5.1–7.0 ft	17	2.3	5	5.6
7.1–9.0 ft	4	.2	0	......
9.1–13.0 ft	14	1.0	9	6.0
13.1–19.0 ft	4	.4	0	......
19.1 or more ft	10	1.6	3	2.5
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)				
Structure narrower by more than 1 foot	15	5.0	0	......
Structure from 1 foot narrower to 1 foot wider	28	4.7	2	3.3
Structure wider by:				
1.1–3.0 ft	70	4.5	6	6.0
3.1–5.0 ft	61	4.1	1	10.0
5.1–7.0 ft	10	1.3	2	2.5
7.1–9.0 ft	4	.2	0	......
9.1–13.0 ft	7	.5	7	5.0
13.1–19.0 ft	4	.5	0	......
19.1 or more ft	10	1.5	2	1.4
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)				
Structure narrower by more than 1 foot	21	5.7	0	......
Structure from 1 foot narrower to 1 foot wider	56	3.6	2	2.9
Structure wider by:				
1.1–3.0 ft	81	4.0	6	5.5
3.1–5.0 ft	87	3.1	2	5.0
5.1–7.0 ft	17	1.3	5	3.1
7.1–9.0 ft	4	.2	0	......
9.1–13.0 ft	14	.6	9	4.5
13.1–19.0 ft	4	.4	0	......
19.1 or more ft	10	1.1	3	1.9

The effect of traffic volume on the accident rate is worth noting. Increased volume reduces the accident rate in most cases. This is the same effect that was noted for two-lane curves. The two effects may possibly be related, since neither of them occurs on roads of more than two lanes.

Table 26 also gives the same information for three-lane roads. The information is meager, but the intersections having less than 10 percent cross traffic appear to be much safer than the others. High volumes are associated with high accident rates.

As with the other roadway types, cross traffic between 10 and 19 percent makes an intersection on a four-lane undivided road much more dangerous than if the cross traffic is below 10 percent. Further increases in cross traffic are less important. The total traffic volume has no consistent effect (see table 27).

On four-lane divided roads without controlled access, the conclusion about cross traffic is again the same. High volume appears to increase the hazard somewhat.

Figure 7 presents the accident rates at intersections for all types of roadways combined. The bars in the chart are of uniform width. In all three volume groups, more than 85 percent of the exposure was at intersections with less than 10 percent cross traffic. The evidence is overwhelming that cross traffic in excess of 10 percent of the total traffic makes an intersection very much more dangerous than if the cross traffic is less than 10 percent.

When all road types are combined, it appears that the accident rate goes up with increases in the total volume. But the exact opposite is the case for intersections on two-lane roads.

Three-way intersections are much safer than four-way intersections on two-lane roads, according to table 28. The angle at which the roads intersect (whether T or Y) does not make any appreciable difference. An increase in the percentage of night traffic reduces the accident rate, as with two-lane tangents.

A breakdown of the effect of commercial traffic, cross traffic, and volume at two-lane intersections is given in table 29. There is some tendency for the accident rate to increase with increasing commercial traffic, but it is not statistically significant.

In summary, the percentage of cross traffic at an intersection is important, and so is the number of approaches to the intersection. Night traffic reduces the accident rate, while the effect of commercial traffic is not clear.

It would be of interest to know how the accident rate is affected by the type of traffic control—stop signs, traffic signals, etc. The question cannot be answered from the data in the present study, because there is too little variety in traffic control at the intersections on the study routes.

Structures

Structures have been classified according to the relative width of the roadway at the structure—on the bridge or in the underpass—as compared with the adjoining pavement. Table 30 presents the accident rates on this basis. The table is restricted to two-lane roads with pavements less than 30 feet wide. Extra width in relation to the approach pavement definitely reduces the accident hazard on bridges. Table 31 shows that for bridges having the same *relative* roadway width, the actual width of the bridge pavement also contributes to the safety of the bridge.

There were not enough underpasses in the study to warrant any conclusions about the effect of roadway width in them. Such evidence as there is, based on a total of 28 accidents, indicates that underpasses are considerably more dangerous than overpasses, even though the average extra width of the underpasses in this study is about 2 feet more than that of the bridges.

Conclusions

The conclusions fall under two headings: (1) a brief summary of the findings, (2) a critique of the study itself. The latter is as important as the former in guiding future research into the cause of accidents.

A summary of the findings was given near the beginning of this report and will not be repeated here. A number of significant relations were discovered, while others that had been expected to be clear-cut did not turn out as expected.

At the beginning of the analysis, the question was raised as to which type of accident rate would prove most reliable of the three types used: (1) adjusted accidents (based on total-to-fatal ratio), all States; (2) actual accidents, selected States (selected for their presumed completeness of reporting); (3) actual accidents, all States. The type 3 rate makes the best showing. Of the first 32 analyses to be made (e.g., two-lane tangents by grade, two-lane tangents by volume, etc.), the type 3 rate behaves credibly in 30 of them and is not seriously misleading in any, though it fails to bring out the effect of traffic volume on two-lane curves. The type 1 rate, in contrast, is reasonable in only 25 of the analyses, and in 2 cases gives results which are significant by statistical tests and yet are seriously misleading. The type 2 rate is misleading in only 1 case, but there are 9 cases in which it is excessively irregular or else contains too little data to give useful results. It would seem that, with the present data at least, one can probably do no better than simply to use all the reported accidents at face value.

The most striking feature of the study is the amount of irregularity in most of the results. Few of the data which have been presented in tables and graphs can be fitted by really smooth curves. There is considerable scatter about the overall trends, and it is likely that some subtle relations are masked by these irregularities.

The fluctuations are much larger than one would expect from considerations of the theory of sampling. They may be due, in part, to errors in the data, such as the failure of the original accident reports to specify the accident locations with sufficient accuracy.

But their principal cause is probably the tremendous complexity of the problem itself. Accidents are associated with so many factors, in such a multitude of combinations, that one has to resort to drastic oversimplifications in order to make any kind of order out of a chaotic mass of material. The remarkable thing is not the irregularities in the tabulations but the number of useful conclusions which do emerge. Most of these conclusions were suspected before the present study was begun, but the statistical analyses give them a broader foundation in hard facts than they ever had before.

However, the foundation is not as broad as it ought to be. Too many of the States were unable to provide information of sufficient accuracy and detail for use in this study. While there has been some improvement in this regard since 1941, the situation is still far from satisfactory. If further progress is to be made in understanding the causes of accidents, many of our States will have to make substantial improvements in the quality of their accident reporting.

Table 31.—Accident rates (type 3) at bridges and overpasses on two-lane roads less than 30 feet wide, by relative width of bridge roadway and adjoining pavement, and actual width of bridge roadway

[*rates are per 10 million vehicles*]

Relative width	Number of accidents and accident rate when width of roadway on bridge is—									
	Less than 20 feet		20 to 24 feet		25 to 29 feet		30 to 34 feet		35 feet or more	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Bridge narrower by more than 1 foot..	17	8.1	4	2.5	0	....	0	....	0	....
From 1 foot narrower to 1 foot wider .	28	5.6	27	2.7	1	0.9	0	....	0	....
Bridge wider by:										
1.1–3.0 ft..	5	25.0	76	4.0	0	....	0	....	0	....
3.1–5.0 ft	2	20.0	76	3.1	7	2.6	2	4.0	0	....
5.1–7.0 ft..	0	....	13	1.6	2	.5	2	2.0	0	....
More than 7 ft	0	....	0	....	4	.2	14	.7	14	0.9
Total . .	52	7.2	196	3.1	14	0.4	18	0.8	14	0.9

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AS OF APRIL 30, 1953

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$13,014	$20,032	$10,333	375.1	$5,793	$2,903	190.8	$34,504	$17,597	466.9	$60,329	$30,833	1,032.8
Arizona	1,101	6,734	4,756	147.7	2,348	1,672	27.4	5,850	3,517	69.4	14,932	9,945	244.5
Arkansas	7,637	9,484	5,060	328.4	5,281	2,662	180.8	14,162	7,211	233.1	28,927	14,933	742.3
California	4,162	32,930	15,910	173.6	18,922	9,867	76.8	92,996	45,370	225.6	144,848	71,147	476.0
Colorado	3,953	11,637	6,595	278.9	2,040	1,131	55.8	11,694	5,856	144.6	25,371	13,582	479.3
Connecticut	7,577	1,023	565	2.4	3,306	1,626	9.2	12,245	6,234	27.2	16,574	8,425	38.8
Delaware	3,563	650	325	.8				7,257	3,671	34.1	7,907	3,996	34.9
Florida	5,008	22,647	11,487	388.7	5,268	2,743	80.0	16,184	8,181	262.1	44,099	22,411	730.8
Georgia	15,085	10,371	5,382	227.6	3,843	1,921	34.0	37,072	17,787	575.5	51,286	25,090	837.1
Idaho	4,357	12,204	7,439	294.5	2,959	1,856	55.9	8,658	5,540	140.0	23,821	14,835	490.4
Illinois	12,220	44,896	24,043	302.2	33,844	16,966	388.7	53,684	28,219	378.6	132,424	69,228	1,069.5
Indiana	14,117	34,715	18,180	155.9	10,102	4,989	48.7	28,199	15,139	228.1	73,016	38,308	432.7
Iowa	3,719	24,606	13,918	668.4	7,131	3,578	318.5	10,131	5,125	450.7	41,868	22,621	1,437.6
Kansas	6,802	15,632	7,635	1,272.4	8,874	4,432	337.9	12,490	6,015	685.5	36,996	18,082	2,295.8
Kentucky	5,839	16,290	8,797	228.9	8,090	4,356	97.6	16,548	8,257	285.0	40,928	21,410	611.5
Louisiana	4,206	16,878	8,467	132.0	10,701	5,323	62.7	22,812	10,920	126.1	50,391	24,710	320.8
Maine	4,459	3,179	1,737	19.2	3,432	1,903	5.4	11,058	5,144	81.2	17,669	8,784	105.8
Maryland	9,126	9,552	4,466	82.4	2,477	1,229	25.7	9,887	5,328	42.0	21,916	11,023	150.1
Massachusetts	11,823	3,602	1,946	12.2	858	429		43,518	20,723	46.5	47,978	23,098	58.7
Michigan	11,365	26,377	13,409	524.2	10,465	5,250	185.3	58,070	25,394	251.5	94,912	44,053	961.0
Minnesota	6,298	18,841	10,074	1,350.3	9,642	5,088	484.2	9,635	5,298	209.9	38,118	20,460	2,044.4
Mississippi	3,421	17,736	8,929	682.4	8,710	4,486	276.9	15,918	8,356	467.9	42,364	21,771	1,427.2
Missouri	12,253	18,115	9,471	817.7	23,903	11,381	200.6	38,396	20,189	447.9	80,414	41,041	1,466.2
Montana	7,620	14,886	9,369	317.9	3,881	2,254	76.1	15,696	9,455	243.4	34,463	21,078	637.4
Nebraska	16,796	10,893	5,951	495.7	2,283	1,696	63.3	11,830	6,005	289.7	25,006	13,652	848.7
Nevada	5,266	4,935	4,131	114.2	2,211	1,812	50.9	5,018	3,862	138.1	12,164	9,805	303.2
New Hampshire	2,827	4,450	2,225	26.4	85	47		5,892	3,069	37.1	10,427	5,341	63.5
New Jersey	7,580	7,242	3,468	36.7	7,741	3,838	5.2	31,923	15,487	35.3	46,906	22,793	77.2
New Mexico	1,675	6,178	3,944	167.3	4,078	2,593	81.1	7,509	4,776	246.3	17,765	11,313	494.7
New York	42,527	81,047	43,286	93.7	27,755	13,843	28.0	113,999	52,805	325.4	222,801	109,934	447.1
North Carolina	6,551	27,834	13,372	635.1	4,575	2,178	169.3	30,690	14,778	499.6	63,099	30,328	1,304.0
North Dakota	2,104	11,398	5,710	1,349.7	6,473	3,377	463.5	5,848	2,961	441.4	23,719	12,048	2,254.6
Ohio	13,535	34,723	16,823	164.8	8,804	4,357	40.8	85,764	42,955	116.7	129,291	64,135	322.3
Oklahoma	13,181	10,156	5,541	168.4	2,562	1,365	47.5	18,011	9,462	288.2	30,729	16,368	504.1
Oregon	3,809	2,285	1,320	58.3	3,186	1,899	41.3	13,766	8,281	195.0	19,237	11,500	294.6
Pennsylvania	14,713	35,927	15,972	97.7	24,233	12,079	90.0	83,658	41,623	176.7	143,818	69,674	364.4
Rhode Island	1,531	3,907	1,953	39.9	1,053	526	3.6	20,049	10,394	28.2	25,009	12,873	71.7
South Carolina	6,862	10,042	5,352	261.9	3,374	1,692	101.6	16,598	8,564	420.2	30,014	15,608	783.7
South Dakota	1,508	11,688	6,885	783.2	4,234	2,403	234.3	7,415	4,572	390.4	23,337	13,860	1,407.9
Tennessee	9,934	13,715	6,670	393.6	2,730	1,371	83.6	36,187	16,269	321.0	52,632	24,310	798.2
Texas	20,595	11,589	6,278	192.0	12,481	6,634	231.0	60,342	32,623	1,247.4	84,412	45,535	1,670.4
Utah	454	6,170	4,713	107.4	3,583	2,694	42.3	9,275	7,036	138.2	19,028	14,443	287.9
Vermont	1,211	5,088	2,728	49.1	1,176	588	7.1	7,530	3,758	49.4	13,794	7,074	105.6
Virginia	[illegible]	15,146	7,228	133.8	8,375	4,158	154.1	30,722	14,799	254.7	54,243	26,185	542.6
Washington	[illegible]	14,288	7,570	241.9	4,163	2,116	79.2	12,998	6,919	70.9	31,449	16,605	392.0
West Virginia	[illegible]	5,768	2,907	30.4	4,405	2,223	10.1	18,851	9,460	155.7	29,024	14,590	196.2
Wisconsin	[illegible]	16,994	9,093	318.5	15,206	7,432	242.6	30,244	15,559	297.4	62,444	32,084	858.5
Wyoming	[illegible]	5,418	3,529	159.8	1,243	824	43.5	7,734	5,124	137.3	14,395	9,477	340.6
[illegible]	[illegible]	[illegible]	1,744	8.6	742	371	3.2	9,328	4,521	19.2	13,662	6,636	31.0
[illegible]	[illegible]	[illegible]	5,054	8.3	4,211	2,104	1.2	7,143	3,281	.7	24,284	11,239	10.2
[illegible]	[illegible]	[illegible]	[illegible]	60.0	1,278	608	6.5	12,218	5,807	42.8	22,258	10,697	109.3
[illegible]	[illegible]	[illegible]	[illegible]	14,980.2	354,110	182,873	5,543.8	1,287,206	649,276	12,485.8	2,416,498	1,238,971	33,009.8

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Bituminous-surfaced highway (U.S. 17) between Georgetown and Myrtle Beach, S. C.

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U. S. DEPARTMENT OF COMMERCE
[illegible] WEEKS, Secretary

BUREAU OF PUBLIC ROADS
[illegible] Commissioner

Studies of the Hardening Properties of Asphaltic Materials

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by JARL T. PAULS,
Chief, Bituminous Section,
and J. YORK WELBORN,
Highway Physical Research Engineer

The rate and amount of hardening of asphaltic materials is an important factor in the service behavior and life of bituminous pavements.

This article describes several methods which can be used for evaluating the hardening properties of asphaltic materials. Two methods, the abrasion test and the weathering-strength test, were developed and used for evaluating the hardening properties of a large number of asphaltic materials. These methods of test are based upon the changes in the physical properties of standard sand-asphalt mixtures during exposure to heat and air. The thin-film oven test, developed by the Bureau of Public Roads some years ago, was also used to evaluate the hardening of these asphaltic materials. Comparisons of the results of the abrasion test, the weathering-strength test, and the thin-film oven test show that the rate and amount of hardening of asphaltic materials can be determined by any of these methods.

From this study it is concluded that:

1. Asphalts differ in their hardening properties, depending on the source of the crude petroleum and the methods used in their manufacture.

2. When subjected to weathering, cracked asphalts become hard and brittle more rapidly than do uncracked asphalts and also develop a higher degree of hardness and brittleness. the rate of hardening increasing with the degree of cracking.

3. Increase in hardness of the asphalt as indicated by decrease in penetration is accompanied by changes in other properties, such as increase in softening point and decrease in ductility.

4. Since the thin-film oven test produces changes in the asphalt comparable to those produced by the other test methods and since it is a more rapid and simpler test to make, it is more suitable for use as a specification test.

AMONG the many factors that may cause the failure of a bituminous pavement, the hardening properties of the bituminous material are highly important. A quantitative knowledge of the tendency of an asphalt to harden is one of the indispensable measures in predicting the service behavior of a bituminous pavement. Investigations have shown that pavement failures increase when the penetration of the asphalt drops to certain critical values, which vary somewhat depending upon such factors as climate, flexibility of base and subgrade, and traffic conditions.

The hardening of an asphalt, as shown by a drop in its penetration, is accompanied by changes in other properties of the material. The test characteristics affected include ductility, softening point, and the amount of material insoluble in selective solvents. Some of these properties often are altered to a greater degree than can be accounted for by the decrease in penetration alone. In this report the changes in the properties of the asphaltic materials are discussed mainly from the standpoint of the decrease in penetration induced by various weathering or exposure conditions. The discussion of other characteristics is secondary, although they may be of equal or greater importance.

Asphalt in thin films may harden very rapidly when subjected to high temperatures. In contrast to the normally slow rate of hardening of the binders that occurs in the finished pavement, the alteration in the test characteristics of asphalts that may occur under certain mixing and laying conditions may be striking. In fact, there are data that show that the loss in penetration occurring in the mixing and laying operation may be greater than the loss subsequently occurring in the pavement over a period of as much as 10 years after construction.

In the mixing operation, because of the relatively large mass of the granular materials, compared to the small mass of the asphalt, the temperatures of such materials at the time they are placed in the mixing box have a greater effect than does the temperature of the asphalt. Heating asphalts in a large mass, such as is done in storage tanks at the mixing plant, does not lower their penetration appreciably. It is only when the asphalt coats the hot stone particles as a thin film in the mixing box that the conditions most favorable to rapid hardening are obtained.

Present information indicates that aggregate temperatures lower than 300°F. at the time of mixing are not particularly damaging to the more normal asphalts. However, many asphalts are adversely affected at only slightly higher temperatures, with rapidly increasing loss in penetration occurring with increasingly higher temperatures. Even when precautions are taken to reduce the damage to the asphalts which occurs in the mixing operations, particularly by keeping the mixing temperature only as high as required to obtain a good coating of the stone particles, the amount of hardening of the asphalt occurring in the mixing and laying operations and in subsequent service may nevertheless differ widely for different asphalts, depending on the source of the crude petroleum from which the asphalt is produced and the methods used in its refining.

Conclusions

From the studies made, it may be concluded that:

1. This investigation substantiates the fact developed by previous investigations that the hardening of asphalts caused by weathering is evidenced by a decrease in penetration and ductility and an increase in softening point.

2. Asphalts differ in their hardening properties, depending on the source of the crude petroleum and on the method of refining.

3. When subjected to weathering, asphalts that have been severely cracked in manufacture harden and develop brittleness more rapidly and to a greater degree than those that are not cracked. The rate of hardening increases with the degree of cracking.

4. The following tests have been found to be useful for obtaining a measure of the resistance of asphalts to heat or weathering as evidenced by rapidity and degree of hardening:

Table 1.—Source of crude and method of refining asphalt cements

Identification and source of crude (or where used)	Method of refining
1. California	Reduction and steam.
2. Do	Do.
3. Do	Do.
4. Venezuela	Pipe still, vacuum.
5. Do	Do.
6. Do	Do.
7. Kansas	Shell still.
8. Do	Do.
9. Do	Do.
10. Do	Do. (highly cracked)
11. Kentucky	Dubbs cracking process (highly cracked).
12. Do	Do.
13. Wyoming	Fire and steam (slightly cracked).
14. Mexican (Panuco)	Steam, Trumble pipe still (slightly cracked).
15. Do	
16. Columbia	Pipe still, vacuum.
17. Venezuela	Air and steam (slightly cracked).
18. Do	Pipe still, vacuum.
19. Arkansas (Smackover)	Vacuum at low temperature.
20. Do	Vacuum, 89 mp. flux.
21. Kentucky and Illinois	Fire and steam, probably blown.
22. Trinidad Lake	Fluxed.
23. Bermudez Lake	Fluxed.
24. California (San Joaquin)	
25. Field sample, Rhode Island	
26. Field sample, Virginia	
27. Venezuela	
28. Kentucky or Illinois	
29. California	
30. Kansas	(Highly cracked).
31. Do	
32. Do	
33. Wyoming	
34. Kansas	
35. Kentucky or Illinois	
36. Field sample, Virginia	
37. California	
39. Field Sample, Rhode Island	
40. Venezuela	
41. Do	
42. Kansas	Pipe still, vacuum.
43. Do	Do.
44. Do	Do.
45. Ohio	
46. Venezuela	
47. Kentucky	

(a) The abrasion test, in which resistance to abrasion of a standard asphalt mixture is inversely proportional to the hardness of the asphalt.

(b) The weathering-strength test, in which increase in compressive strength of a standard asphaltic mixture is generally proportional to the hardness of the asphalt.

(c) The Shattuck test, in which the hardness characteristics of the asphalt in a standard mixture are determined after the mixture has been exposed to heat in a standard manner.

(d) Outdoor exposure, in which the hardness characteristics of the asphalt contained in a standard asphaltic mixture are determined after exposure of the mixture to natural weathering.

(e) The thin-film oven test, in which the hardness characteristics of the asphalt are determined after it has been exposed to heat in an oven.

5. With any of the above-mentioned methods of test the same degree of hardening can be obtained by varying the time or temperature of exposure.

6. Of the test methods enumerated above, the thin-film oven test is the most suitable for use as a specification test because it requires the least amount of time, because the hardening of the asphalt is measured directly (rather than indirectly as in the case of the abrasion test and weathering-strength test), and because the test does not require the extraction and recovery of the asphalt, as in the case of the Shattuck test and outdoor exposure tests, and thus eliminates any uncertainty regarding the effect of the recovery procedure on the characteristics of the weathered asphalt.

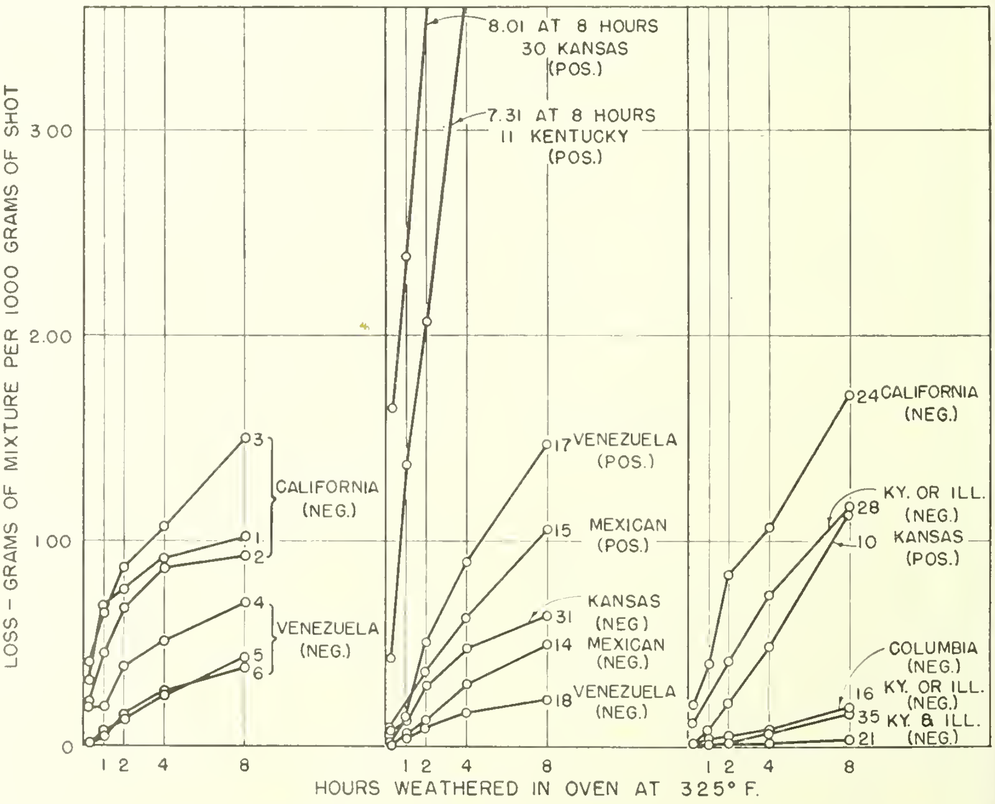

Figure 1.—Results of abrasion test on asphalt cements weathered at 325°F.

Tests of Hardening Properties

The standard test now used for determining the hardening properties of asphalts is the test for loss on heating and the penetration test on the residue. In this test, a 50-gram sample of asphalt is placed in a metal container of such dimensions that the sample has a surface area of about 3.7 square inches and a depth of about thirteen-sixteenths of an inch. The sample is heated in an oven at 325°F. for 5 hours, after which the percentage loss of material and the loss in penetration are determined. It is generally agreed that, because of the relatively large depth of the sample with correspondingly small surface area, the conditions of this test are not sufficiently severe to evaluate the relative hardening properties of asphalt.

Another test of loss on heating is the thin-film oven test,[1] which appears to be a very

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[1] *The properties of the residues of 50-60 and 85-100 penetration asphalts from oven tests and exposure*, by R. H. Lewis and J. Y. Welborn. PUBLIC ROADS, vol. 22, No. 2, April 1941. *Behavior of asphalts in thin-film oven test*, by R. H. Lewis and W. J. Halstead. PUBLIC ROADS, vol. 24, No. 8, April-May-June 1946.

Table 2.—Results of abrasion test on asphalt cements after oven weathering at 325° F.

Identification and source of asphalt	Tests on the original asphalt			Abrasion loss [1] after oven-weathering at 325° F. for—					
	Oliensis spot test	Xylene equivalent	Penetration at 77° F.	None	¼ hour	1 hour	2 hours	4 hours	8 hours
				Gms.	*Gms.*	*Gms.*	*Gms.*	*Gms.*	*Gms.*
1. California	Negative	....	54	3.34	0.32	0 68	0.76	0.91	1 02
2. Do	...do	....	90	1.86	.21	.45	.67	.87	.93
3. Do	...do	....	131	1.45	.41	.65	.87	1.07	1.50
4. Venezuela	...do	....	51	1.17	18	.18	.38	.51	70
5. Do	...do	....	88	.95	.01	.07	13	25	.43
6. Do	..do	....	138	.18	.01	05	.15	.26	.39
7. Kansas	...do	....	61	1.36	.02	04	.20	.40	.51
8. Do	...do	....	83	....	....	....	....	25	.30
9. Do	...do	....	137	.05	.00	....	.02	.07	.20
10. Do	Positive	60–64	92	.99	.02	.08	.21	.49	1 14
11. Kentucky	..do	80–84	87	3 25	43	1.37	2.07	5.54	7.31
12. Do	...do	80–84	102	2.75	.53	1.18	2.41	5.87	8.19
13. Wyoming	...do	20–24	95	1.50	12	.27	.42	.73	1.27
14. Mexican	Negative	....	106	.19	.00	.05	.13	.31	.50
15. Do	Positive	24–28	101	1.38	.09	.19	.37	.63	1.06
16. Columbia	Negative	....	110	.17	02	.04	.06	.08	.19
17. Venezuela	Positive	8–12	97	.77	.02	.14	.51	.90	1.47
18. Do	Negative	....	105	.05	.01	.04	.09	17	.23
19. Arkansas	...do	....	96	.79	.11	.15	.20	.36	.55
20. Do	...do	....	98	.09	00	02	.03	.07	.18
21. Kentucky and Illinois	...do.[2]	....	105	15	00	.01	.02	.01	.03
22. Trinidad Lake	....	....	100	.83	.25	.56	.81	1.05	1.18
23. Bermudez Lake	....	....	102	.41	....	.27	.29	.51	85
24. California	Negative	....	89	2.40	.21	.51	.84	1.07	1.71
25. Field sample, Rhode Island	...do	....	65	1.40	.03	.13	.16	.28	.56
26. Field sample, Virginia	...do	....	89	.37	02	.07	.16	.28	.33
27. Venezuela	...do	....	64	.63	.01	.04	.10	.35	.76
28. Kentucky or Illinois	...do	....	93	1.06	.12	.26	.42	.74	1.17
29. California	...do	....	102	2.01	.15	.36	.70	.86	.92
30. Kansas	Positive	88+	75	3.93	1 65	2.38	3.57	4.92	8.01
31. Do	Negative	....	75	.61	.08	.13	.30	.48	.64
32. Do	..do	....	76	1.32	.46	.49	.67	.97	1.36
33. Wyoming	..do	....	85	1.07	.23	.49	.62	.77	1.13
34. Kansas	..do	....	87	1.01	.03	.10	.31	.33	.66
35. Kentucky or Illinois	...do	....	....	.19	.01	.02	.03	.07	.17
36. Field sample, Virginia	...do	....	67	59	.18	25	.86	1.11	1.66
37. California	...do	....	100	1.91	.23	.65	.81	1.03	1 34
39. Field sample. Rhode Island	...do	....	67	....	....	....	....	....	....
40. Venezuela	...do	....	86	1.24	08	.38	.53	.75	.96
41. Do	...do	....	67	1 61	.61	.34	.57	.80	1 14
42. Kansas	...do	....	68	.35	05	.05	.08	.23	.32
43. Do	...do	....	90	1.02	.49	.27	.46	.59	.90
44. Do	...do	....	132	33	.07	.03	.09	21	48
45. Ohio	...do	....	94	.23	.03	.02	.03	.16	.57
46. Venezuela	...do	....	54	1 53	.28	.25	.24	.38	.96
47. Kentucky	...do	....	89	1.45	50	.30	.28	.38	.62

[1] Loss in grams of mixture per 1,000 grams of shot. [4] Probably blown.

satisfactory method for evaluating the hardening properties of asphalts. In this test a 50-milliliter (approximately 50 gram) sample is placed in a tin 5.5 inches in diameter. Thus, its surface area is nearly 24 square inches and its depth is approximately one-eighth inch. As in the standard test, the sample is heated in an oven at 325°F. for 5 hours, after which the penetration, softening point, and ductility are determined.

Some investigators have preferred to study the effect of heat on asphaltic materials by subjecting aggregate mixtures containing these asphalts to some type of heat test, and then testing either the mixture by some physical test or the extracted and recovered asphalt for penetration, ductility, and softening point.

Two types of tests, both based on the changes occurring in the physical properties of standard mixtures after exposure to heat and air, were developed and used for this study. These were the abrasion test, which is similar to that developed by H. N. Hveem,[2] and the weathering-strength test.

The Abrasion Test

The abrasion test,[3] as made by the Bureau of Public Roads laboratory, was originally

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[2] *Abrasion test*, by H. N. Hveem. California Highway and Public Works, Nov.-Dec. 1946.

[3] *Evaluation of asphalts by means of an abrasion test*, by J. T. Pauls and R. A. Peck. Proceedings of the Association of Asphalt Paving Technologists, vol. 19, 1950.

Table 3.—Results of abrasion test on liquid asphalts after oven weathering at 325° F.

Identification and source of asphaltic material	Tests on the distillation residue				Abrasion loss [1] after oven-weathering at 325° F. for—					
	Spot test	Penetration at 77° F.	Ductility at 77° F.	Softening point	None	¼ hour	1 hour	2 hours	4 hours	8 hours
		Cm.	*Cm.*	*° F.*	*Gms.*	*Gms.*	*Gms.*	*Gms.*	*Gms.*	*Gms.*
L-1. Kansas, RC-4	Negative	78	170	123	4 76	0.36	1.17	1.42	1.81	2.32
L-2. Do	....do	82	130	124	3.07	.59	.73	1.31	1.59	1.96
L-3. Do	Positive [2]	95	132	114	9.03	.32	2.11	3.55	4.80	6.26
L-4. Kansas, MC-4	Negative	196	90	124	2.12	10	.13	.21	.46	1.02
L-5. Do	....do	246	70	126	.33	.01	.03	.10	.18	.36
L-6. Do	Positive [2]	158	80	126	2.46	.11	1.05	2.40	3.82	6.12
L-7. Michigan, SC-4	Negative	....	....	....	....	.00	.02	.05	.14	.68
L-8. Do	Positive [2]	....	....	....	....	.37	3.19	11.30	23.95	34.25

[1] Loss in grams of mixture per 1,000 grams of shot. [2] Highly cracked.

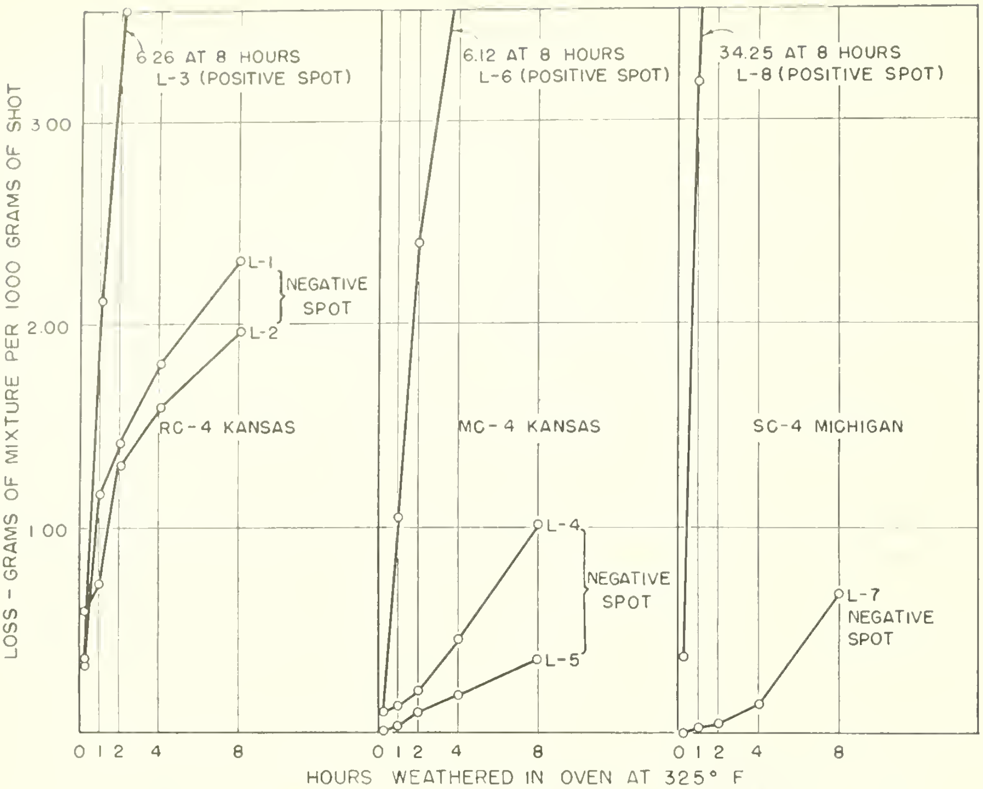

Figure 2.—Results of abrasion test on liquid asphalts weathered at 325°F.

described in a paper presented at the meeting of the Association of Asphalt Paving Technologists at St. Louis in 1950. The present paper includes the data from the original paper and also the results of additional work done subsequent to that time.

Essentially, the abrasion test consists of the determination of the loss in weight of an asphaltic mixture resulting from dropping a definite quantity of shot from a height of 1 meter onto the revolving horizontal surface of the weathered test specimen. It is assumed that the amount of loss, from the abrasive action of the shot, is directly related to the hardness of the asphalt.

In the California procedure developed by Hveem, molded mixtures of Ottawa sand and asphalt are weathered in a special weathering machine which employs an air temperature of 140°F. and direct rays of standard drying lamps, which emit the bulk of their energy in the infrared band. One cycle requires 5 hours of exposure and the complete test consists of nine cycles. In the Public Roads procedure, the test specimens are weathered in an oven at 325°F. for various periods of time from 15 minutes to 8 hours.

The important difference between the Public Roads test and the California test is in the weathering of the specimens. Oven weathering at 325°F., the procedure used in this study, is more rapid and if, as is believed, it evaluates the hardening properties of the materials properly, it has this advantage over the less severe and more time-consuming procedure used by Hveem.

Another difference in procedure is in the temperature at which the specimen is held while being tested. In the Public Roads laboratory the test is made at 77°F., whereas California uses 60°F. One would expect somewhat higher abrasion losses with the lower test temperature, other test conditions being the same. Exact control of the test temperature is most important in making this test because any change or difference in temperature quickly affects the consistency of the asphalt film on the surface of the sand particles and, consequently, the abrasion loss.

Abrasion losses were determined for oven-weathering periods ranging from 15 minutes to 8 hours, and for longer periods in a few instances. Weathering at a temperature of 325°F. for periods longer than 8 hours appeared to have no particular value although, as would be expected, the abrasion loss increased with an increase in the duration of the oven exposure.

Uniform oven weathering of the test specimens was not obtained in the early test work. Using a forced-draft type oven, the degree of weathering was found to differ considerably depending on the position of the individual test specimen in the oven. This variation in weathering was largely eliminated by placing the test specimens in an oven pan slightly deeper than the

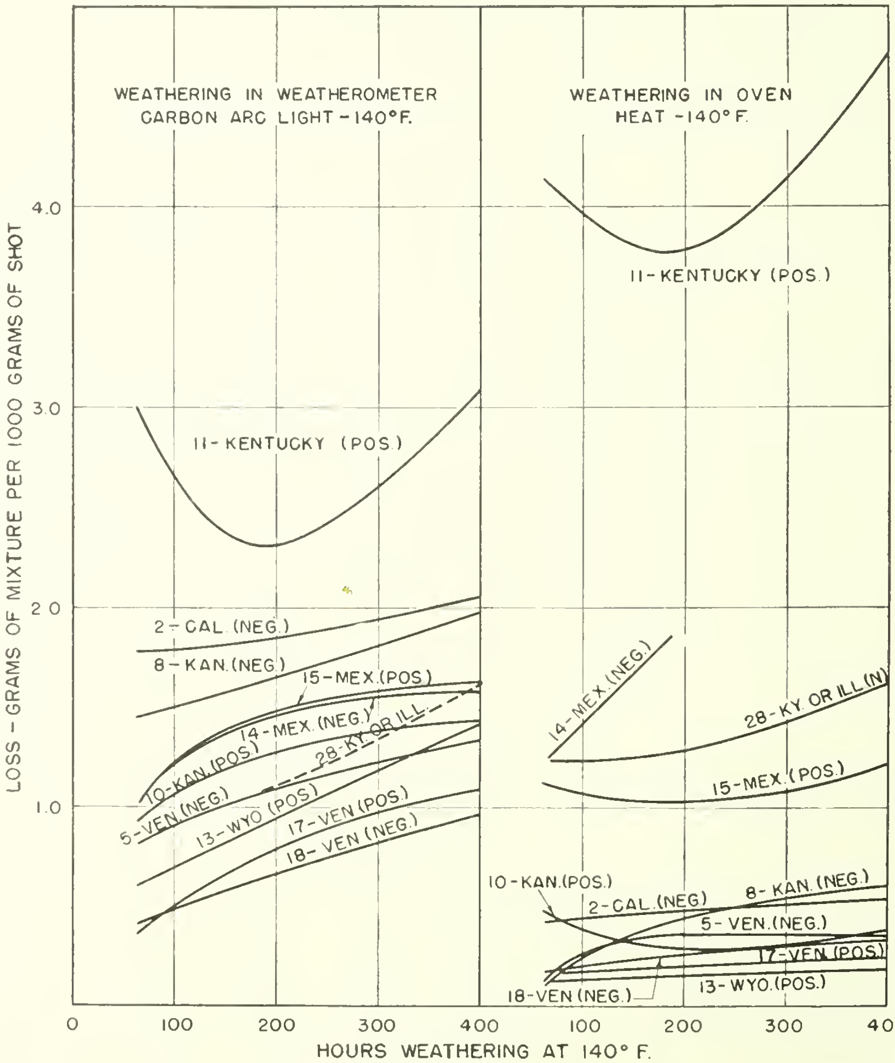

Figure 3.—Results of abrasion test on asphalt cements weathered at 140° F.

Table 4.—Comparative abrasion losses of asphalt cements after weathering at low temperature in oven and in weatherometer at 140° F.

Identification and source of asphalt	Abrasion loss [1] after weathering at 140° F. for—					
	64 hours		192 hours		400 hours	
	Weather-ometer	Oven	Weather-ometer	Oven	Weather-ometer	Oven
2. California	1.79	0.43	1.85	0.48	2.07	0.54
5. Venezuela	.82	.14	1.09	.36	1.33	.36
8. Kansas	1.45	.11	1.63	.44	1.99	.61
10. Kansas (highly cracked)	.94	.49	1.29	.30	1.43	.39
11. Kentucky (highly cracked)	3.09	4.14	2.31	3.78	3.10	4.79
13. Wyoming (slightly cracked)	.62	.13	.93	.16	1.43	.19
14. Mexican	1.01	1.24	1.46	1.86	1.58	......
15. Mexican (slightly cracked)	1.01	1.13	1.48	1.03	1.63	1.22
17. Venezuela (slightly cracked)	.37	.17	.77	.18	1.09	.25
18. Venezuela	.42	.18	.66	.24	.97	.34
28. Kentucky or Illinois	.85	1.24	1.11	1.28	1.62	1.62

[1] Loss in grams of mixture per 1,000 grams of shot.

height of the specimens and covering the pan with a fine wire screen, thus reducing the air currents around the specimens. Very uniform weathering conditions were obtained by following this procedure.

In order to make a direct comparison between the result of slow and rapid weathering, two limited series of tests were made in which the weathering temperature was 140°F. One of these utilized a conventional weatherometer with carbon-arc lamps and controlled ventilators for maintaining the desired temperature, and the other series was weathered in an electric oven set to maintain a temperature of 140°F. The results of these comparative tests are included in this report. The details of the abrasion test are described on page 199.

The Weathering-Strength Test

The weathering-strength test developed and used in this study is based on the findings of previous investigations that the compressive strength on cylindrical specimens of bituminous mixtures, composed of Ottawa sand and asphalt, when tested in compression without lateral support, is primarily a measure of the cohesive strength of the asphalt, a property closely related to its penetration or hardness. Accordingly, the test procedure consists of exposing molded cylindrical specimens of Ottawa sand and asphalt mixtures in an oven at 325°F. for various periods of time and determining their compressive strengths.

The difficulty of weathering molded cylindrical specimens of asphaltic mixtures at a temperature of 325°F. without having them deform was overcome by placing the test specimens in perforated oversize cans and filling the space between the specimen and the can with Ottawa sand. The support provided by the sand prevented the test specimens from deforming in all instances, even when weathering mixtures of low strength such as those composed of liquid asphalts. The details of the weathering-strength tests are given on page 201.

A considerable number of asphaltic materials were tested by the abrasion test and by the weathering-strength test in this study. They cover a range in type, grade, source, and method of refining. The source and method of refining are not known for a number of these materials and, in some instances, the only identification available is the location in which they were used. Table 1 lists the asphalt cements which were tested, with such information relative to the source of the crude and the method of refining as is available.

Abrasion Test on Asphalt Cements

Test results on the original asphalt cements and the results of the abrasion test are given in table 2.

Generally low abrasion losses were obtained on all the negative-spot asphalts prepared from the midcontinent petroleums. Of the materials from this source, No. 21, made from Kentucky-Illinois petroleum and reported as probably blown, had the lowest loss in abrasion, 0.03 grams, after 8 hours of oven weathering. More typical abrasion losses on materials from this field were those obtained from Arkansas No. 20, Kansas No. 31, and Kentucky or Illinois No. 35. They were 0.18, 0.64 and 0.17 grams, respectively. The highest losses for the negative-spot materials from the midcontinent petroleum area were 1.17 for Kentucky or Illinois No. 28, and 1.36 for Kansas No. 32.

Very high losses were obtained on the highly cracked midcontinent materials Nos. 11 and 12 from Kentucky, and No. 30 from Kansas. The loss on No. 11 after 8 hours of weathering was at the rate of 7.31 grams, on No. 12, 8.19 grams, and on No. 30, 8.01

Table 5.—Weathering-strength test results on asphalt cements

Identification and source of asphalt	Tests on original asphalt		Compressive strength at 77° F. after weathering for—					
	Oliensis spot test	Penetration at 77° F.	1 hour	2 hours	3 hours	4 hours	6 hours	8 hours
			P.s.i.	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*
1. California	Negative	54	72	136	199	264	323	351
2. Do	do	90	41	84	....	208	....	352
3. Do	do	131	24	53	129	176	222	257
4. Venezuela	do	51	63	107	....	169	....	209
5. Do	do	88	34	54	....	118	....	191
6. Do	do	138	21	37	58	89	123	145
7. Kansas	do	61	62	88	115	129	142	144
8. Do	do	83	40	69	....	103	....	117
9. Do	do	137	19	37	49	76	100	111
10. Do	Positive	92	14	18	20	21	22	29
11. Kentucky	do	87	115	111	45	46	50	47
12. Do	do	102	115	111	45	45	....	46
13. Wyoming	do	95	33	63	....	108	....	116
14. Mexican	Negative	106	33	60	....	120	....	180
15. Do	Positive	101	31	64	....	93	....	131
16. Columbia	Negative	110	21	43	....	105	....	162
17. Venezuela	Positive	97	49	73	110	122	....	146
18. Do	Negative	105	29	48	78	98	....	156
19. Arkansas	do	96	18	33	50	63	96	119
20. Do	do	98	18	30	48	55	89	97
21. Kentucky and Illinois [1]	do	105	14	25	....	44	....	62
22. Trinidad Lake	....	100	41	51	89	132	151	191
23. Bermudez Lake	....	102	47	65	....	170	190	210
24. California	Negative	89	46	94	169	241	290	319
25. Field sample, Rhode Island	do	65	32	59	87	112	146	191
26. Field sample, Virginia	do	89	25	42	62	90	132	158
27. Venezuela	do	64	60	86	118	135	152	174
28. Kentucky and Illinois	do	93	38	72	....	115	....	138
29. California	do	102	31	100	....	189	....	342
30. Kansas	Positive	75	128	118	....	92	....	102
31. Do	Negative	75	43	74	....	104	....	132
32. Do	do	76	29	59	....	116	....	149
33. Wyoming	do	85	27	48	....	122	....	144
34. Kansas	do	87	29	54	....	95	....	132
35. Kentucky or Illinois	do	89	26	44	....	75	....	123
36. Field sample, Virginia	do	67	47	89	....	169	....	250
37. California	do	100	20	54	....	158	....	284
39. Field sample, Rhode Island	do	67	29	63	....	152	....	237
40. Venezuela	do	86	36	66	....	118	....	169
41. Do	do	67	24	36	....	74	....	138
42. Kansas	do	68	39	65	....	106	....	154
43. Do	do	90	16	23	....	55	....	117
44. Do	do	132	15	28	....	58	....	100
45. Ohio	do	94	18	36	....	68	....	101
46. Venezuela	do	54	44	65	....	108	....	177
47. Kentucky	do	89	19	32	....	58	....	115

[1] Probably blown.

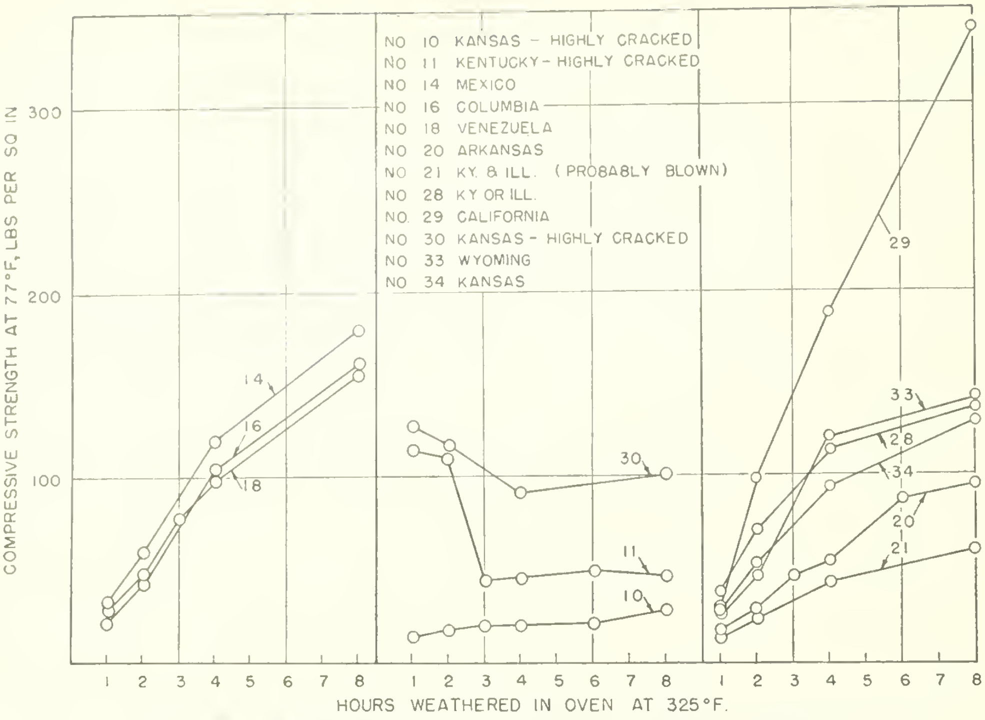

Figure 4.—Weathering-compressive strength test results, asphalt cements.

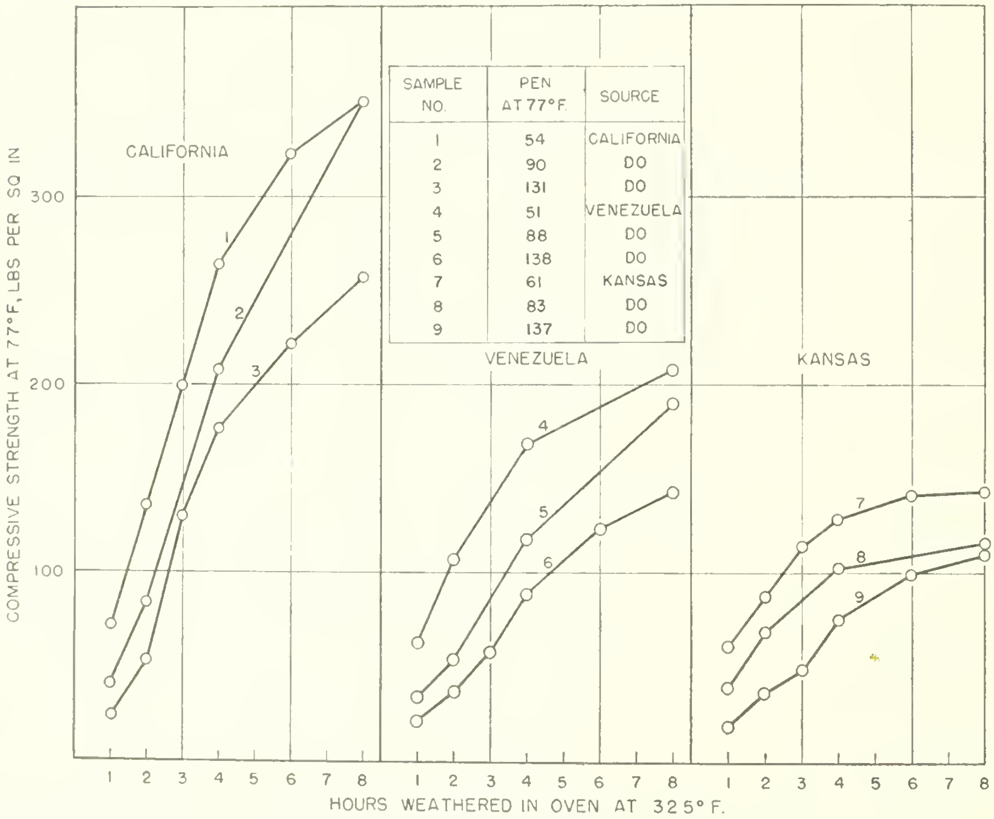

Figure 5.—Results of weathering-compressive strength test showing effect of source of crude and the penetration of the original asphalt.

grams. The loss on the highly cracked material, No. 30, may be compared with the loss of only 0.64 grams on No. 31, a negative-spot material from the same source. The loss on Nos. 11 and 12 may be compared with the loss of only 0.17 on No. 35, and a loss of 0.62 on No. 47, negative-spot materials from a similar crude.

From sources other than midcontinent fields the results are not so consistent. The Trinidad and Bermudez asphalts and the negative-spot California asphalts had comparatively high abrasion losses, while some of the other negative-spot foreign and domestic asphalts had relatively low losses. The two slightly cracked materials, showing positive spots—No. 15 from Mexico and No. 17 from Venezuela—had high abrasion losses as compared with the negative-spot materials No. 14 and No. 18 from the same respective crude oils. Typical differences between negative- and positive-spot materials are shown in figure 1.

There is some indication that, in general, the abrasion losses are somewhat higher for the more viscous grades of asphalts from a given crude than for the softer materials. This is shown by the abrasion losses obtained on the Venezuela materials Nos. 4, 5, and 6, and the Kansas materials Nos. 7, 8, and 9. In the California group, materials Nos. 1, 2, and 3, the No. 3 asphalt, the least viscous of the three, does not conform to this general relation as the abrasion loss for this material is somewhat higher than that for the more viscous materials, Nos. 1 and 2, from the same source.

Abrasion Test on Liquid Asphaltic Materials

The results of abrasion tests on a limited number of liquid asphaltic materials are given in table 3. The same relation between positive- and negative-spot materials with respect to abrasion loss was obtained with cutbacks and slow-curing asphaltic materials as was found in the results of tests on the paving asphalts. A positive-spot *RC*–4 cutback (No. L–3) made from a Kansas crude had a loss of 6.26 grams after 8 hours of weathering compared with losses of 2.32 and 1.96 grams on two negative-spot *RC*–4 materials (Nos. L–1 and L–2) from the same source. In the case of *MC*–4 materials from Kansas, the positive-spot material No. L–6 had a loss of 6.12 grams while the negative-spot materials, Nos. L–4 and L–5, had losses of only 1.02 and 0.36 grams, respectively. No. L–7, a negative-spot *SC*–4 material from Michigan, had a loss of only 0.68 grams whereas No. L–8, a positive-spot *SC*–4 material from the same source, had a loss of 34.25 grams. These relations are shown graphically in figure 2.

Losses from Slow Weathering

As stated before, some work has been done to correlate the abrasion losses from mixtures weathered rapidly at 325°F., with those from mixtures weathered slowly at 140°F. Two different methods were used in the slow-weathering study: (1) Conventional weatherometer with carbon-arc lamps, a rotating shelf, and an air-control system to maintain the interior temperature at 140°F.; and (2) an electric oven with mechanical convection and temperature control.

The abrasion losses resulting from weathering in the oven at 140°F. and those resulting from exposure in the weatherometer are given in table 4 and plotted in figure 3. These comparative tests were made on 11 materials. Weathering periods of 64, 192, and 400 hours were used.

In general, higher abrasion losses resulted from weathering in the weatherometer than from weathering in the oven at 140°F. No. 11, a severely cracked material, was an exception in that oven weathering at 140°F. caused very much higher abrasion loss than did weathering in the weatherometer. Also, in the case of No. 14, a negative-spot material from Mexico, somewhat higher loss occurred in the oven than in the weatherometer after 192 hours of exposure. In the case of material No. 28, oven weathering produced a higher loss than did the weatherometer at 64 hours and 192 hours, but the losses by the two methods of weathering were the same at 400 hours of weathering.

Less satisfactory differentiation between cracked and uncracked materials was obtained by weathering at 140°F. in the weatherometer than by weathering in the oven at 325°F. For the positive-spot material No. 17 and the negative-spot material No. 18, both from the same source, the losses in abrasion for oven weathering at 325°F. for 8 hours were 1.47 and 0.23 grams, respectively (see table 2); while for weathering in the weatherometer for 400 hours, the losses were 1.09 and 0.97 grams (table 4).

Weathering in this particular type of weatherometer results in a rapid surface-hardening effect on the test specimens with very slow hardening below the immediate surface. The rate of abrasion loss at the immediate surface would therefore probably be very high, while the rate of loss from the mixture below the surface would be comparatively low.

Slow weathering in the oven at 140°F., up to 400 hours, also failed to differentiate satisfactorily between the cracked and non-cracked materials. Only in the case of No. 11, a very highly cracked material, is this property of the material definitely shown. With the Venezuelan asphalts, the positive-spot material No. 17 showed a lower loss (0.25 grams) after 400 hours in the 140°F. oven than did the negative-spot material No. 18, (0.34 grams) after the same exposure. Indications are that weathering in the low temperature oven is not severe enough to bring out differences in the hardening properties of the asphalts within a reasonable period of weathering.

The following is a summary of the abrasion test results as discussed above:

1. In general, comparatively low abrasion losses were obtained on the negative-spot asphalts prepared from the midcontinent petroleums.

2. Of the negative-spot domestic asphalts included in this investigation, the California materials gave, in general, the higher abrasion losses.

3. The positive-spot asphalts were found to have high abrasion losses compared to the non-cracked materials from the same source. Generally, the amount of loss varied directly with the degree of cracking.

4. On the basis of results of the abrasion test, a direct comparison of materials from different sources is not warranted, as the abrasion loss of a negative-spot material from one source may be higher than that of a positive-spot material from a different source. On the other hand, the abrasion test appears to indicate quite accurately differences in quality that are due to various manufacturing procedures where comparisons are between materials from the same source.

5. The results of the abrasion test made with specimens weathered in an oven at 325°F. are more informative than when the specimens are weathered in an oven or in a weatherometer at 140°F.

The Weathering-Strength Test

Compressive strengths of the weathered specimens are given in table 5. The effect of time of weathering at 325°F. on the compressive strength of the molded specimens is shown in figures 4 and 5. Table 6 and figure 6 give similar data for the liquid asphaltic materials included in this study.

The compressive strength of the test specimens, except those containing highly cracked asphalt, increased with increased time of weathering up to 8 hours, the longest period of weathering used. The test specimens containing the highly cracked asphalts, Nos. 11, 12, and 30, decreased in strength after the first hour of weathering until, after weathering for 3 or 4 hours, they reached a substantially constant strength. The only other highly cracked asphalt cement tested, No. 10 from Kansas, differed from these in that it gave considerably lower strength results and showed some slight increase in strength with increased weathering. Mixtures containing the asphalts that showed positive spots but were only slightly cracked, Nos. 13, 15, and 17, showed increase in strength with increased exposure. No. 20, an uncracked asphalt from Arkansas, showed a relatively low increase in strength with weathering as did No. 21 from Kentucky and Illinois, a material reported as probably blown.

For the uncracked materials, the source of the crude from which the asphalt was obtained is shown to have a major effect on the rate of increase in strength on weathering of the mixture. The California

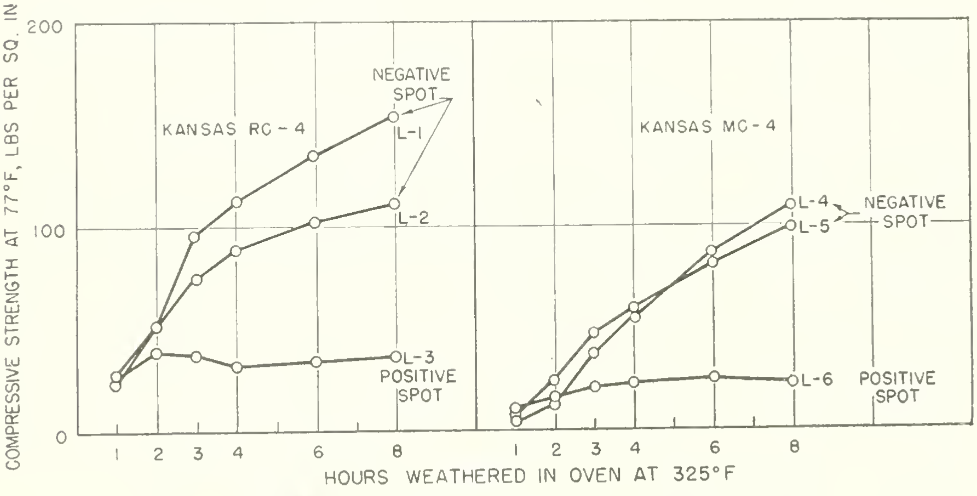

Figure 6.—Weathering-strength test results, cutback asphalts.

Table 6.—Results of weathering-strength test on cutback asphalts

Identification and source of asphaltic material	Tests on the distillation residue							Compressive strength at 77° F. after weathering for—					
	Oliensis spot test	Penetration at 77° F.	Ductility at 77° F.	Softening point	Residue from thin-film oven test			1 hour	2 hours	3 hours	4 hours	6 hours	8 hours
					Penetration at 77° F.	Ductility at 77° F.	Softening point						
			Cm.	*° F.*		*Cm.*	*° F.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P s.i.*	*P.s.i.*	*P.s.i.*
L-1. Kansas, RC-4	Negative	78	170	123	46	90	137	23	51	95	112	135	154
L-2. Do	do	82	130	124	46	15	144	26	51	75	89	102	111
L-3. Do	Positive [1]	95	132	114	33	42	134	27	39	37	32	34	36
L-4. Kansas, MC-4	Negative	196	90	124	74	164	124	3	13	36	55	86	109
L-5. Do	do	246	70	126	90	81	126	7	24	47	59	81	99
L-6. Do	Positive [1]	158	80	126	51	99	126	10	15	20	22	24	22

[1] Highly cracked.

Table 7.—Tests on asphalt recovered from oven-weathered cylinders,[1] and abrasion and compressive-strength tests of molded specimens

Identification and source of asphalt (and original penetration)	Time of weathering	Tests on asphalt recovered from weathered cylinders			Tests on molded specimens after oven-weathering at 325° F.	
		Penetration at 77° F.	Ductility at 77° F.	Softening point	Abrasion loss	Compressive strength at 77° F.
	Hours		*Cm.*	*°F.*	*Grams*	*Lbs./sq. in*
2. California (90 pen.)	1	40	250+	138	0.45	41
	2	19	93	151	67	84
	4	10	5	184	.87	208
	8	4	0	228	.93	352
5. Venezuela (88 pen.)	1	39	37	141	07	34
	2	25	7	162	.13	54
	4	17	3	192	.25	118
	8	11	2	250	.43	191
8 Kansas (83 pen.)	1	36	7	152	....	40
	2	26	4	180	....	69
	4	20	3	212	.25	103
	8	14	1	267	.30	117
14. Mexican (106 pen.)	1	39	23	148	.05	33
	2	29	8	165	.13	60
	4	20	4	187	.31	120
	8	11	0	260	.50	180
15. Mexican (101 pen.)	1	34	5	160	.19	31
	2	24	3	189	.37	64
	4	16	2	249	.63	93
	8	9	1	325	1.06	131
21. Kentucky and Illinois (105 pen.)	1	70	37	132	.01	14
	2	58	14	140	.02	25
	4	40	5	160	.01	44
	8	31	3	190	.03	62
28. Kentucky or Illinois (93 pen.)	1	39	48	139	.26	38
	2	23	6	159	.42	72
	4	17	4	179	.74	115
	8	....	....	....	1.17	138
29. California (102 pen.)	1	33	140	133	.36	31
	2	17	17	151	.70	100
	4	10	4	178	.86	189
30. Kansas (75 pen.) (highly cracked)	1	12	5	160	2.38	128
	2	8	1	190	3.57	118
	4	4	0	251	4.92	92
31. Kansas (75 pen.)	1	32	5	160	.13	43
	2	23	3	186	.30	74
	4	17	2	217	.48	104
32. Kansas (76 pen.)	1	43	18	140	.49	29
	2	29	5	160	.67	59
	4	21	3	185	.97	116
33. Wyoming (85 pen.)	1	37	16	141	.49	27
	2	27	6	155	.62	48
	4	18	3	184	.77	122
34. Kansas (87 pen.)	1	40	10	146	.10	29
	2	28	4	165	.31	54
	4	21	3	191	.33	95

[1] 2- by 2-inch molded cylinders oven-weathered at 325° F.

asphalts, Nos. 1, 2, 3, 24, 29, and 37, had the highest rate of increase in strength during weathering and the highest strength after 8 hours of weathering. The asphalts from some of the midcontinent fields gave comparatively low strength values after 8 hours, while the foreign materials gave strength values between those of the midcontinent and California asphalts.

The strength curves differ with differences in penetration of the original asphalt. In general, the more viscous materials from a given source gave somewhat higher strengths after weathering than did the less viscous ones. This is illustrated in figure 5 where the strength of the groups of California, Venezuela, and Kansas asphalts each show increased strength with decreasing penetration.

In table 6 and figure 6 are shown the weathering-strength test results on mixtures containing cutback asphalts *MC*–4 and *RC*–4. Very low weathering-strength curves were obtained for the highly cracked materials L–3 and L–6 as compared to the uncracked materials.

The following is a summary of the weathering-strength test results as discussed above:

1. Mixtures containing uncracked asphalts increased in strength for increasing periods of oven weathering up to 8 hours, the maximum period of weathering used in this investigation.

2. There was a wide difference in the rate of increase of strength and in the strength after 8 hours of oven weathering for asphalts from the various sources.

3. Three mixtures containing cracked asphalts attained relatively high strength in the early weathering periods and then decreased in strength as the weathering period was increased to 3 or 4 hours. A fourth cracked asphalt with a somewhat lower zylene equivalent (Kansas No. 10) had a low strength after 1 hour of weathering and thereafter showed only slight increase in strength for longer periods of weathering.

It has been shown that the abrasion loss and the strength of molded Ottawa sand and asphalt mixtures, when weathered for various periods of time, differ widely depending on the source of the asphalt and the method used in its manufacture. In order to determine if this difference in abrasion loss and strength is a measure of the difference in hardness of the contained asphalts, the following study was made.

Tests on Recovered Asphalts

In order to determine the changes in the characteristics of the asphalts after various periods of weathering, a number of the asphalts included in this study were extracted and recovered from test specimens weathered by the same method used in the compressive-strength test. To do this, a number of specimens were made for each weathering period to allow extraction and recovery of a sufficient amount of the asphalt to permit determining the penetration, ductility, and softening point of the materials at the various weathering periods. The Abson test method was used in recovering the asphalt. The results of tests on the recovered asphalts are given in table 7, together with the results of the abrasion and compressive-strength tests for the same periods of weathering.

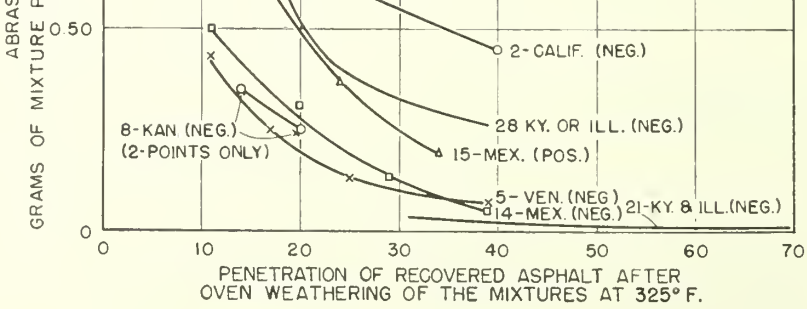

Figure 7.—Relation of the loss in the abrasion test to the penetration of the asphalts recovered from oven-weathered, 2 by 2-inch cylindrical specimens.

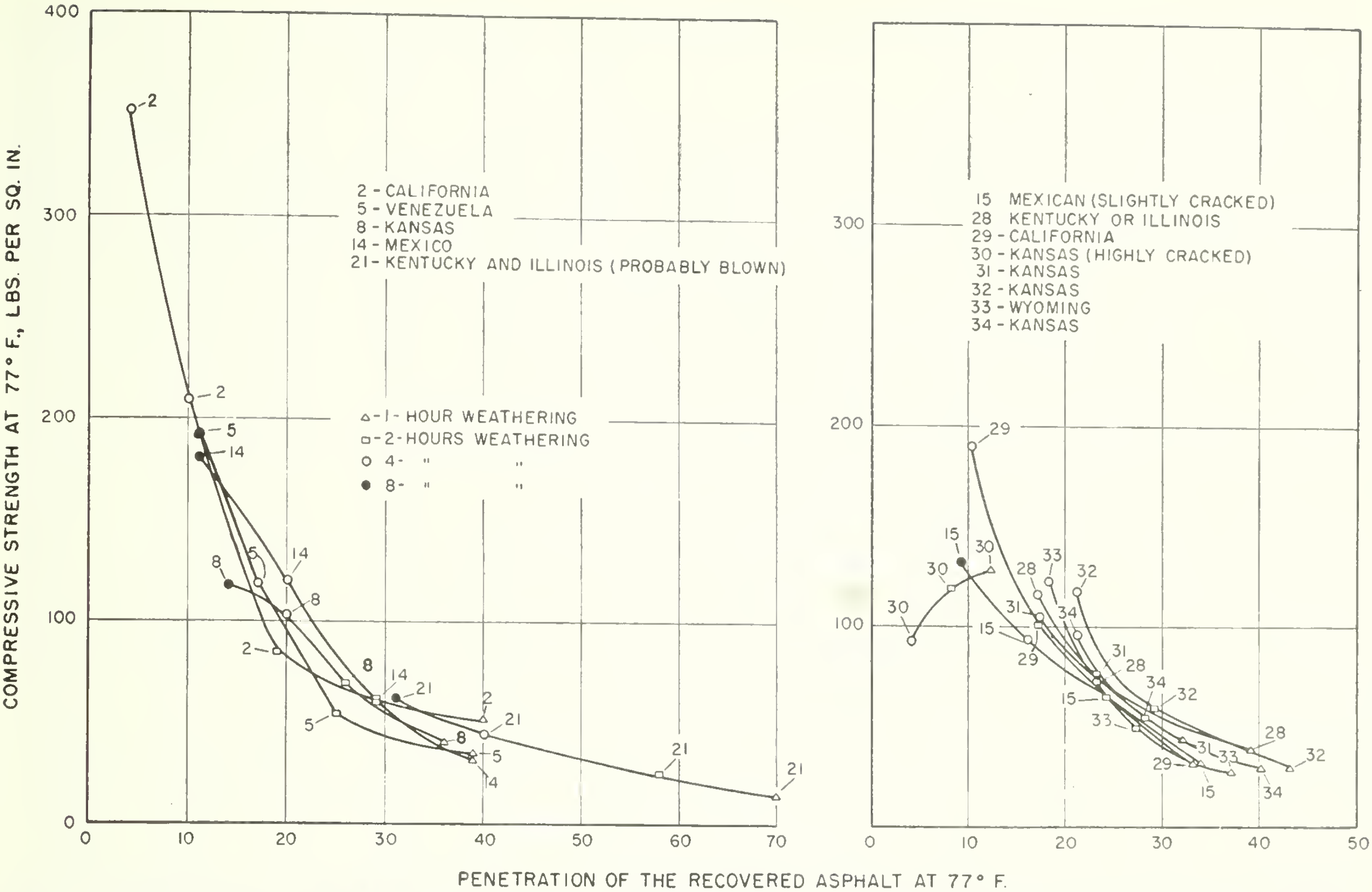

Figure 8.—Weathering-compressive strength test results. Relation between compressive strength and penetration of the recovered asphalt.

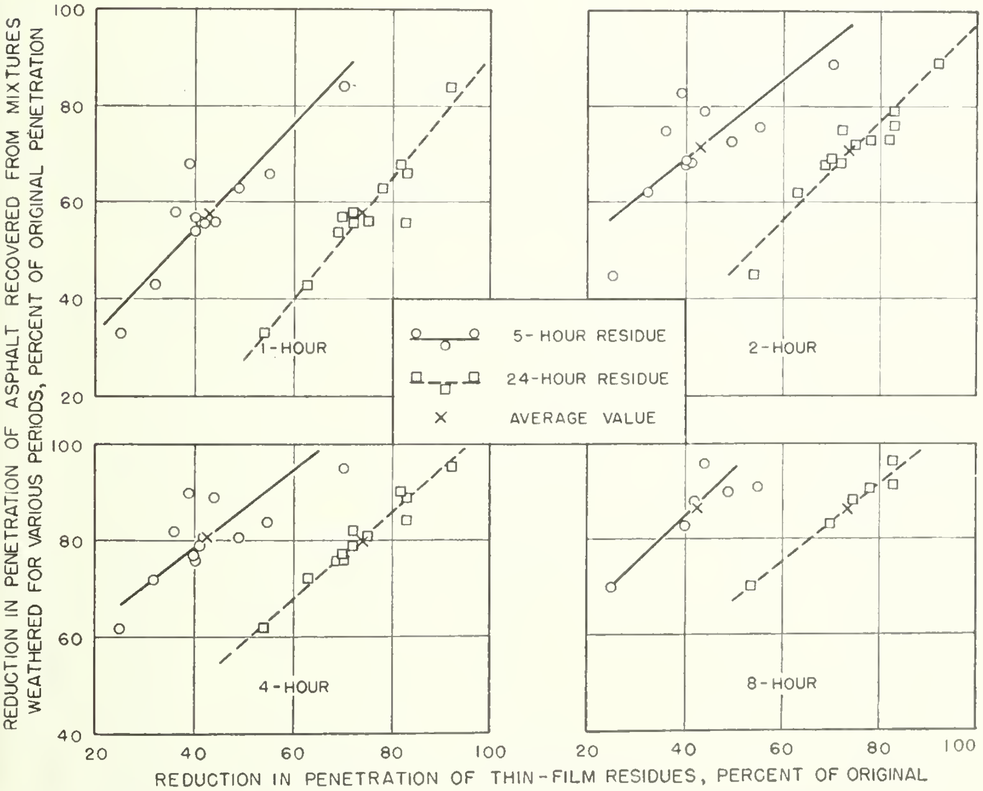

Figure 9.—Comparison of the reduction in penetration of thin-film residues with the reduction in penetration of asphalts recovered from oven-weathered mixtures.

On account of the differences in dimensions of the specimens, it is not to be expected that the amount of hardening that will occur in a given time in the weathering-strength specimens will be the same as that which occurs in the abrasion specimens. However, it is reasonable to assume that the rate of hardening will follow the same general trends in both types of test.

In figure 7 is shown, for typical materials, the relation between abrasion loss and the penetration of the asphalt recovered from the weathering-strength specimens. Table 7 shows that the hardness of the recovered asphalt—that is, the reduction in penetration and increase in softening point, increases with length of weathering. Both table 7 and figure 7 show that the abrasion loss increases as the penetration of the recovered asphalt decreases. The Kentucky and Illinois asphalt, No. 21, which had the least abrasion loss of all the materials tested, also showed the least hardening as evidenced by the penetration and softening point of the recovered asphalt. Other materials, such as Kansas No. 8, Venezuelan No. 5, and Mexican No. 14, which had low abrasion losses, also showed low rates of hardening up to 8 hours of weathering. In the case of the highly cracked asphalt No. 30, the asphalt hardened rapidly and had very high losses in the abrasion test.

It is evident that the abrasion loss is related to the consistency of the contained asphalt and that either may be used to determine the relative resistance of asphaltic materials to hardening.

Effect of Consistency

Figure 8 shows the compressive strength of the weathered specimens plotted against

Table 8.—Results of thin-film oven tests on asphalt cements

Identification and source of asphalt	Tests on original asphalt			Tests on residue from thin-film test, ⅛-inch film exposed at 325° F.							
				5-hour test				24-hour test			
	Penetration at 77° F.	Softening point	Ductility at 77° F.	Penetration at 77° F.	Retained penetration [1]	Softening point	Ductility at 77° F.	Penetration at 77° F.	Retained penetration [1]	Softening point	Ductility at 77° F.
		° F.	*Cm.*		*Percent*	*° F.*	*Cm.*		*Percent*	*° F.*	*Cm.*
1. California	54	119	250+	33	61	128	250+				
2. Do	90	112	200	51	56	121	215	16	17	147	134
3. Do	131	107	150	68	52	118	200				
4. Venezuela	51	130	230	33	65	143	63				
5. Do	88	118	250+	51	58	132	242	22	25	171	5
6. Do	138	110	152	74	54	123	207				
7. Kansas	61	129	130	40	65	146	12				
8. Do	83	121	200	50	60	138	25	25	30	183	3
9. Do	137	110	154	73	53	127	105				
10. Do	92	115	115	60	65	141	10				
11. Kentucky	87	113	250+	28	28	139	24				
12. Do	102	111	170	28	27	142	15				
13. Wyoming	95	115	180	50	52	135	76				
14. Mexican	106	117	199	54	51	136	113	24	22	174	6
15. Do	101	118	106	46	45	143	12	18	17	219	3
16. Columbia	110	112	151	66	60	124	210				
17. Venezuela	97	117	165	50	51	140	28				
18. Do	105	114	140	62	60	128	140				
19. Arkansas	96	113	142	63	65	126	234				
20. Do	98	121	170	61	63	135	58				
21. Kentucky and Illinois	105	119	120	79	75	127	67	48	46	148	10
22. Trinidad	100	112	247	47	47	126	45				
23. Bermudez	102	112	58	33	27	126	28				
24. California	89	111	150+	47	52	136	230				
25. Field sample, Rhode Island	65	120	250+	40	61	134	250+				
26. Field sample, Virginia	89	114	214	60	67	127	237+				
27. Venezuela	64	125	220	40	62	141	90				
28. Kentucky or Illinois	93	116	154	60	64	127	235	26	28	157	8
29. California	102	111	203	62	61	119	250+	18	18	149	45
30. Kansas	75	115	158	22	30	137	47	6	8	191	0
31. Do	75	123	198	45	60	141	18	23	30	187	3
32. Do	76	122	142	52	68	133	121	28	37	166	5
33. Wyoming	85	119	186	50	59	132	170	24	28	165	5
34. Kansas	87	119	170	52	60	133	100	27	31	170	4
35. Kentucky or Illinois	89	122	188	63	71	132	127				
41. Venezuela	67	118	235	54	61	130	135				
42. Kansas	68	124	193	42	62	143	21				
43. Do	90	117	183	55	61	131	76				
44. Do	132	113	180	70	53	127	74				
45. Ohio	94	120	195	56	60	134	52				
46. Venezuela	54	129	250+	34	63	137	112				
47. Kentucky	89	118	187	54	61	128	195				

[1] Percent of original penetration.

the penetration of the contained asphalt after various periods of exposure. It is of interest to note that, in general, the points for most of the asphalts are close to one curve, indicating that the compressive strength varies inversely as the penetration of the contained asphalt. An exception to this is the highly cracked asphalt No. 30. At the end of 1 hour of weathering the penetration of this asphalt had been reduced to 12 from an original value of 75 and further weathering reduced both the penetration and strength. The highly cracked asphalts, No. 11 and No. 12, were not recovered from the weathered mixtures but it is probable that for these materials the trend of the relation between retained penetration and strength would be similar to that for asphalt No. 30.

The slightly cracked asphalt, No. 15, gave increased strength as the penetration decreased, although the rate of increase was slightly lower than for the uncracked materials. Of the uncracked materials, California asphalt No. 2 had the lowest penetration and the highest strength after 8 hours exposure, these being 4 and 352, respectively. Asphalt No. 30, a highly cracked asphalt, had a penetration of 4 after 4 hours exposure but had a strength of only 92 pounds per square inch. This would indicate that the highly cracked materials become brittle or fragile and lose their cohesive properties to a greater extent than do the uncracked materials.

Table 9.—Comparison of the penetrations of asphalts after the thin-film oven test with the penetrations of the same asphalts recovered from mixtures after weathering

Identification and source of asphalt	Oliensis spot test	Penetration at 77° F.			
		Original asphalt	Residue from thin-film oven test [1]	Recovered asphalt 1-year outdoor weathering	Recovered asphalt 22-hours ultra-violet light at 180° F.
A. Mexican	Negative	87	24	31	24
B. Midcontinent	Positive	98	8	15	11
C. Mexican	Negative	86	23	29	25
D. Midcontinent	do	90	30	35	32
E. Do	Slightly positive	85	25	33	29
F. Arkansas	Positive	81	15	19	22
G. Do	Negative	90	31	36	36
H. Texas	Slightly positive	87	19	20	21
J. Do	Negative	86	21	23	24
K. Mexican	do	80	23	29	24
N. Illinois	do	95	40	51	52
O. Arkansas	do	91	28	32	35
P. California	do	87	14	20	21
Q. Wyoming	do	90	17	28	28
R. Do	Slightly positive	91	17	30	29
S. California	Negative	93	12	23	16
T. Wyoming	do	83	13	21	18
U. Do	Positive	82	5	10	8
V. Kansas	Negative	92	24	28	30
W. Do	Positive	86	14	20	19
X. Kentucky	do	86	8	16	12

[1] ⅛-inch film exposed at 325° F. for 25 hours.

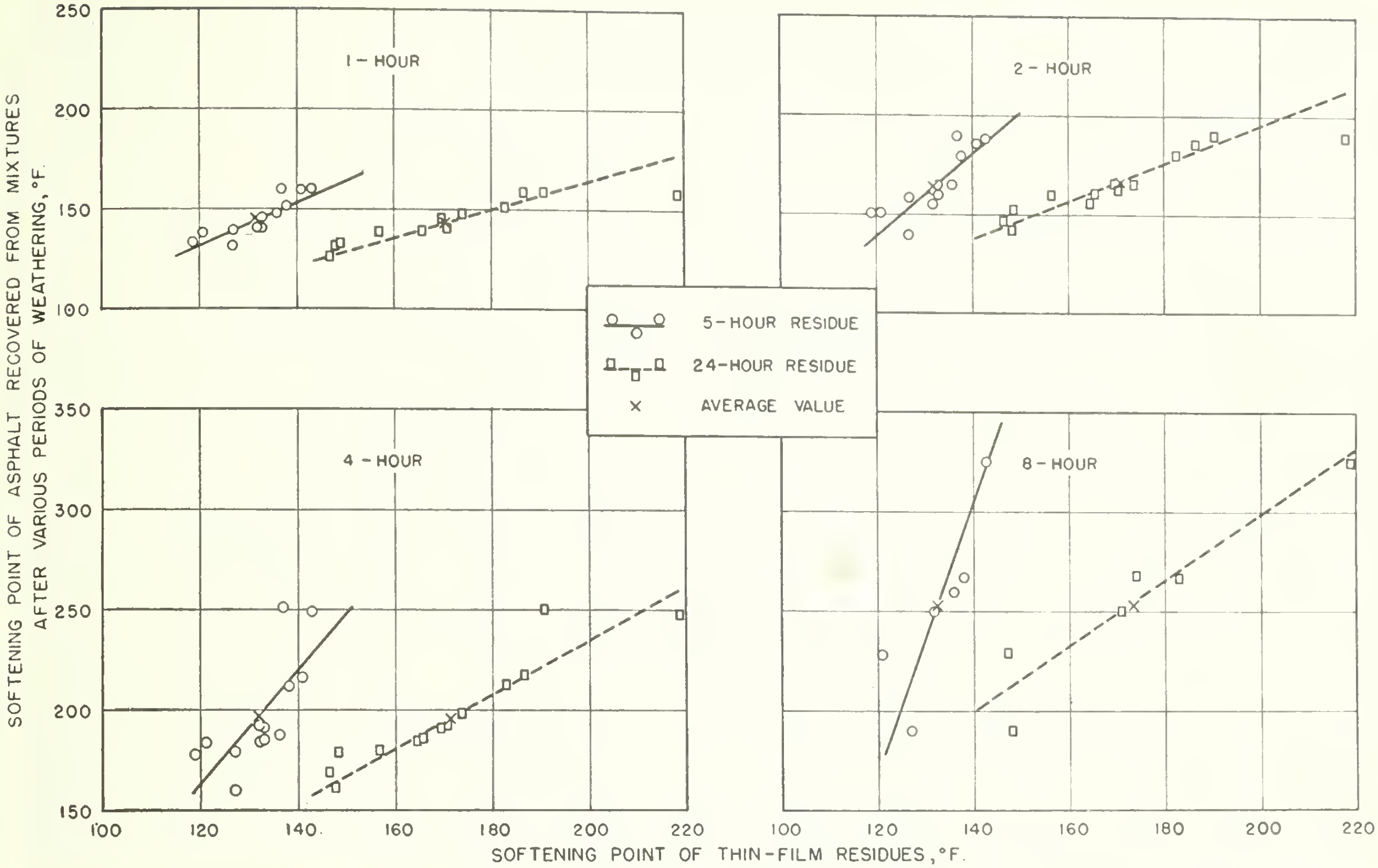

***Figure 10.**—Comparison of softening point of thin-film residues with the softening point of asphalts recovered from oven-weathered mixtures.*

There is a wide difference in the amount and rate of hardening of the various asphalts included in this study. Some of the midcontinent asphalts had higher resistance to hardening than did some of the asphalts from the higher asphaltic-base crudes. The retained percentages of original penetration after 1 and 8 hours of weathering were 44 and 4, respectively, for California No. 2, 44 and 12 for Venezuela No. 5, 43 and 17 for Kansas No. 8, 37 and 10 for Mexican No. 14, and 67 and 29 for Kentucky and Illinois No. 21.

After weathering 1 and 4 hours, the percentage of original penetration retained by some of the other asphalts were as follows: For Kansas No. 30, a highly cracked material, 16 and 5, respectively; for uncracked Kansas material No. 32, 57 and 28, respectively; and for Mexican No. 15, slightly cracked, 34 and 16.

The results of a previous study[4] show that the physical properties of a given mix-

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[4] *The effect of characteristics of asphalts on physical properties of bituminous mixtures,* by R. H. Lewis and J. Y. Welborn. PUBLIC ROADS, vol. 25, No. 5, Sept. 1948.

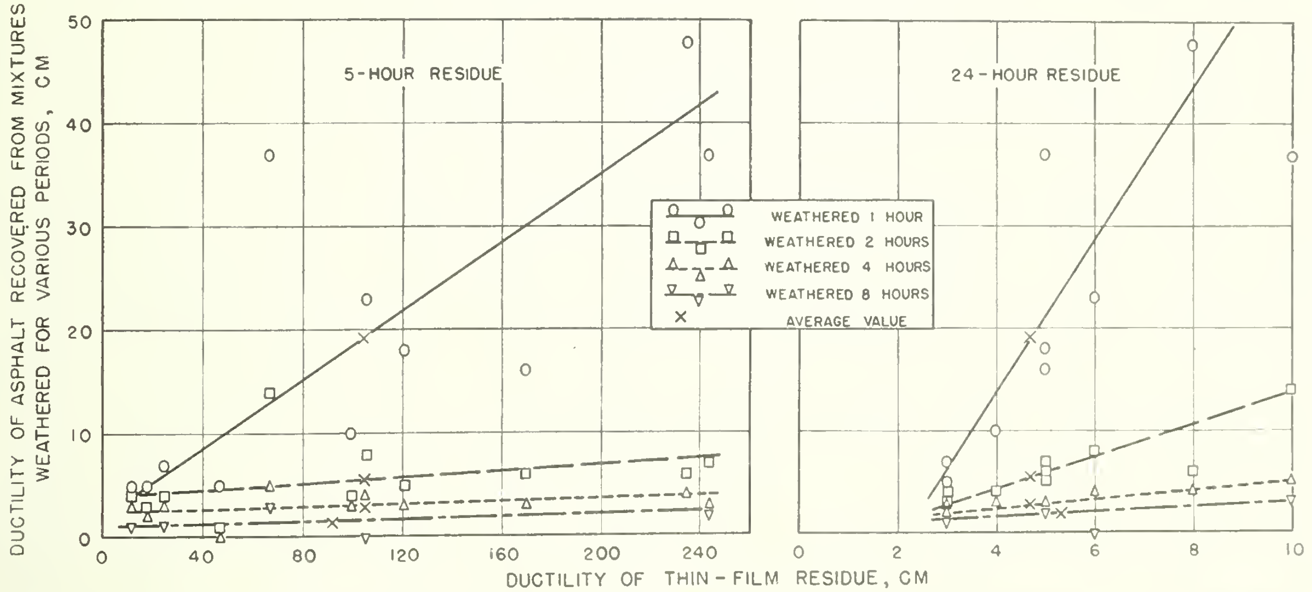

***Figure 11.**—Comparison of ductility of thin-film residues with the ductility of the asphalts recovered from oven-weathered mixtures.*

ture are dependent upon the consistency of the contained asphalt at the test temperature. The results of the present work substantiates these earlier findings.

The results of the tests on the recovered asphalts from the test cylinders after weathering are summarized as follows:

1. There is a wide difference in the resistance of asphalts to hardening depending on the source of the crude petroleum from which the asphalt is obtained and the method of refining.
2. In general, the abrasion loss is dependent upon the consistency of the contained asphalt.
3. For the uncracked asphalts and those only slightly cracked, the compressive strength of the mixtures varied inversely as the penetration of the contained asphalt without regard to the source of the asphalts.
4. For the highly cracked asphalts, the relation between the compressive strength and the hardness of the contained asphalt is different than for the uncracked asphalts.

Behavior Dependent on Hardening

The foregoing studies of the abrasion and weathering-strength tests clearly show that the behavior of asphaltic materials in these tests is dependent largely upon the rate and amount of hardening that occurs during the exposure of Ottawa sand and asphalt mixtures to heat and air. Since both tests are time-consuming and require the control of many variables involved in their test procedures, they are not well suited for use as specification tests. Previous studies of the hardening properties of asphaltic materials have shown that the thin-film oven test also can be used to show the rate of change in asphaltic materials when subjected to heat and air. Therefore, a study has been made to determine the correlation that may exist between the results of this test and the results of the abrasion and weathering-strength tests.

Thin-Film Oven Tests

Thin-film oven tests were made on most of the asphalts used in this study in order to compare the characteristics of the residues obtained by this method of direct exposure of the asphalts to the characteristics of the asphalts after exposure in mixtures. The results of the thin-film oven tests are given in table 8, together with the test results on the original asphalts. In most of the previous work, the thin-film oven test has been made on a 50-milliliter sample of asphalt in an aluminum pan 5.5 inches in diameter, the film being exposed in an oven for 5 hours at 325°F. The same control is used as for the standard test for loss on heating. In order to produce greater changes in the asphalts by the thin-film oven test, which would be more comparable to the changes occurring during weathering of the mixtures, some of the asphalts used in this study were also exposed in the thin-film oven test for 24 hours. A detailed description of the method used for the thin-film oven test is given on page 202.

Comparisons of the reduction in penetration of the 5-hour and the 24-hour thin-film residues with the reduction in penetration of the asphalts recovered from the Ottawa sand and asphalt mixtures weathered for 1, 2, 4, and 8 hours are shown in figure 9, which presents the reduction in penetration as a percentage of the original penetration by the two methods of exposure. There is some dispersion of the points from the average curves for the 5-hour thin-film residues which may be accounted for by the wide difference in the severity of the two methods of weathering. The points for the 1-hour weathering period fall fairly close to the straight line, indicating that the amount of hardening produced by the two methods of exposure are in the same order. The comparision for the 24-hour thin-film residues in general is better than for the 5-hour thin-film test. It is of interest to note that the percentage reduction in the penetration of the asphalt from the 24-hour thin-film oven test is almost identical with the percentage reduction in penetration of the asphalt recovered from the mixture after weathering for 2 hours. It is apparent from the above comparisons that both methods of exposure give comparable results as to the hardening properties of asphalts.

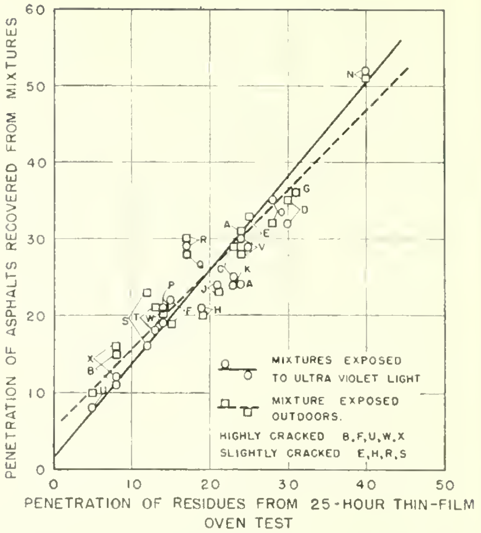

Figure 12.—Comparison of penetration of asphalt recovered from mixtures exposed to ultraviolet light and outdoors with the penetration of the thin-film oven test.

Comparisons of the softening points of the residues after the 5-hour and the 24-hour thin-film oven tests with those of the asphalts recovered from the weathered mixtures are shown in figure 10. As for the penetration data, this figure shows that changes in the asphalts as measured by the softening point, in general, are in the same order for the two methods of exposure. Here also the softening points of the residues from the 24-hour thin-film oven test are approximately the same as for the same

Table 10.—Comparison of the residues from the thin-film oven test with the recovered asphalts from the Shattuck mixing test

Identification and source of asphalt	Penetration of original asphalt at at 77° F.	Residue from thin-film oven test[1]				Recovered asphalt from Shattuck mixing test			
		Penetration at 77° F.	Retained penetration[2]	Softening point	Ductility at 77° F.	Penetration at 77° F.	Retained penetration[2]	Softening point	Ductility at 77° F.
RESULTS OF FIRST SERIES[3]									
			percent	° *F.*	*Cm.*		*percent*	° *F.*	*Cm.*
3. California	54	34	63	129	250+	25	46	135	250+
4. West Texas	52	37	71	138	77	36	71	138	66
6. Columbia	56	35	63	141	105	30	54	145	21
6A. Venezuela	54	35	65	141	145	27	50	150	20
8. East Texas	51	33	65	154	15	34	67	151	17
10A. Venezuela	58	33	57	146	62	22	38	158	7
12. Unknown	51	36	71	150	10	34	67	153	7
12B. Do	51	34	67	128	250+	29	57	130	222
RESULTS OF SECOND SERIES									
1. West Texas	55	43	78	135	158	40	73	135	60
2. Unknown	50	38	76	137	121	34	68	140	44
3. Do	53	37	70	133	244	38	72	134	148
4. West Texas	91	67	74	122	173	69	76	120	181
5. Do	90	62	69	124	250+	63	70	123	185
6. Venezuela	55	37	67	133	218	35	64	138	204
7. Do	84	53	59	129	168	44	49	135	102
8. California	57	37	65	125	250+	37	65	126	250+
9. Do	90	54	60	120	250+	47	52	122	250+

[1] ⅛-inch film exposed at 325° F. for 5 hours.

[2] Percent of original penetration.

[3] Reported in PUBLIC ROADS, vol. 22, No. 2, April 1941.

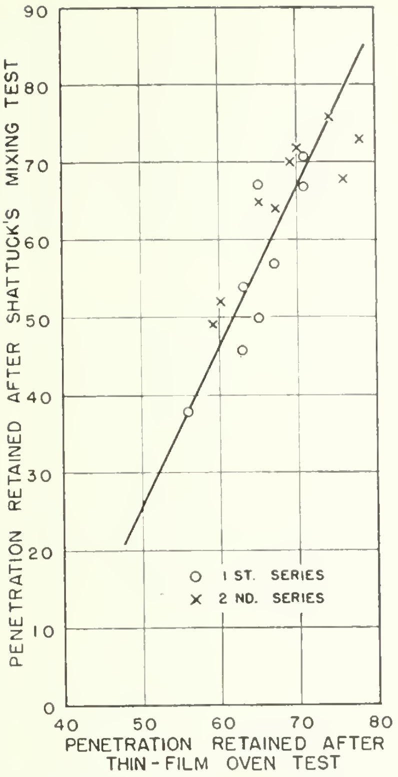

Figure 13. — Comparison of the penetration retained after the thin-film oven test and the Shattuck mixing test.

asphalts recovered from the mixtures weathered for 2 hours.

Figure 11 compares the ductility of the 5-hour and 24-hour thin-film residues with the ductility of the asphalts recovered from the weathered mixtures. Because of the inherent high ductility of the California asphalts, Nos. 2 and 29, and their abnormally large change in ductility with a small change in penetration, their values were omitted. With the exception of those asphalts, figure 11 shows a similarity in changes in ductility resulting from the two methods of exposure.

The above comparisons of penetration, softening point, and ductility of the asphalts recovered from the oven-weathered mixtures and those obtained from the thin-film oven test, show that the heating of asphalts in relatively thin film, as in the thin-film oven test, produces changes in their characteristics similar to those produced by exposing asphaltic mixtures to the same temperature.

Comparison with Other Tests

Lang and Thomas[5] studied the hardening properties of asphalt cements of the 85-100 penetration grade by exposing mixtures of Ottawa sand and these asphalts to the action of ultraviolet light and heat and to natural outdoor weathering.

In the exposure to ultraviolet light, the procedure was as follows: 3,000 grams of prepared Ottawa sand and asphalt mixture were placed in a twin pug-mill mixer which rotated at a speed of 160 r.p.m. for 22 hours. During this time the mixture was exposed to ultraviolet light from a mercury arc in quartz. The temperature of the mixture was maintained at 180°F. during the exposure period.

For the outdoor weathering, similar mixtures were exposed for a period of 1 year. At the completion of the exposure periods the asphalts were extracted and recovered from the weathered mixtures by the Abson method.

Lang and Thomas reported the results of thin-film oven tests made by the method which has just been described with a time of exposure of 25 hours. The results of the tests on the asphalts recovered from the mixtures exposed to ultraviolet light and outdoors and the results of the 25-hour thin-film oven test are given in table 9.

Comparison between the penetration of the thin-film residues and the penetration of the asphalts recovered from mixtures after exposure to ultraviolet light and after weathering outdoors is shown in figure 12.

The thin-film test for 25 hours exposure is shown, in general, to be somewhat more severe than either of the two methods of weathering used on the mixtures. All three test procedures show similar differences in the hardening properties of asphalts as influenced by the source of the material and whether or not, or to what degree, they have been cracked in the refining processes. The authors concluded from these studies that the hardening produced by natural weathering is very similar to that produced by artificial heat, air, and ultraviolet light exposure.

Another type of test for determining the relative resistance of asphalts to hardening is the Shattuck mixing test.[6] In this test, 1,880 grams of heated Ottawa sand are mixed with 120 grams of the asphalt to be tested for 1 minute in a laboratory pug-mill mixer under standard conditions of temperature. After mixing, the mixture is spread evenly in a shallow pan which is then placed in an oven at 350°F. for 30 minutes. The mixture is then removed and allowed to cool to room temperature. After cooling, the asphalt is extracted (using benzol) and recovered by the Abson method. The recovered asphalt is then tested for penetration, ductility, and softening point.

In a previous investigation[7] a study was made of the Shattuck test and it was found that the alterations in the asphalt which take place in this test are very similar to those that take place in the 5-hour, thin-film oven test. The results of a more recent study are shown in table 10, in which results obtained with the Shattuck mixing test and the 5-hour, thin-film oven test are compared. It may be noted in figure 13 that the two methods of test gave similar results.

ABRASION TEST PROCEDURE

Preparing the mixture

Standard Ottawa sand (No. 20–No. 30), as specified in standard method of test for tensile strength of hydraulic-cement mortars, A.S.T.M. Designation C 190, is used. In testing penetration grade asphalts, the sand is heated to 325°F. in an oven before mixing, while for tests on liquid asphaltic materials the sand is dried but not heated. Sufficient asphaltic material for each batch is heated to the application temperature while being stirred to prevent local overheating. The asphaltic material and Ottawa sand are combined in the proportion of 2 parts of asphaltic material to 100 parts by weight of sand.

Application temperatures used for the various asphaltic materials are:

Penetration grades	325°F.
Rapid-curing (RC), medium curing (MC), and slow-curing (SC) liquid asphaltic materials	200°F.
Emulsion	125°F.

Each test mixture is prepared in a batch of sufficient size to produce 15 test specimens. In testing asphalt cement the batch, after mixing, is spread out in shallow pans to a depth of about one-half inch and allowed to cool to air temperature. The mixer and the cooling pans are shown in figure 14. After cooling, 30 grams of the mixture is placed in each of fifteen 2⅜-inch ointment tins, each of which is numbered serially. These tins are then placed in an oven at 325°F. to warm the mix for molding.

The tins, each with its 30 grams of mixture, are placed in the oven one at a time at 3-minute intervals. Molding is started after 9 minutes and continued at the rate of one specimen each 3 minutes so that the pre-heating time for each 30-gram lot is uniformly 9 minutes.

In testing liquid asphaltic materials, the procedure is modified by a preliminary curing of the loose mixture in the large shallow pans in an oven at 140°F. for 18 hours. The

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[5] *Laboratory studies of asphalt cements*, by Fred C. Lang and T. W. Thomas. University of Minnesota Engineering Experiment Station Bulletin No. 15, Nov. 1939.

[6] *Measurement of the resistance of oil asphalts (50-60 pen.) to changes in penetration and ductility at plant mixing temperatures*, by C. L. Shattuck. Proceedings of The Association of Asphalt Paving Technologists, Vol. 11, 1940.

[7] First reference, footnote 1, p. 188.

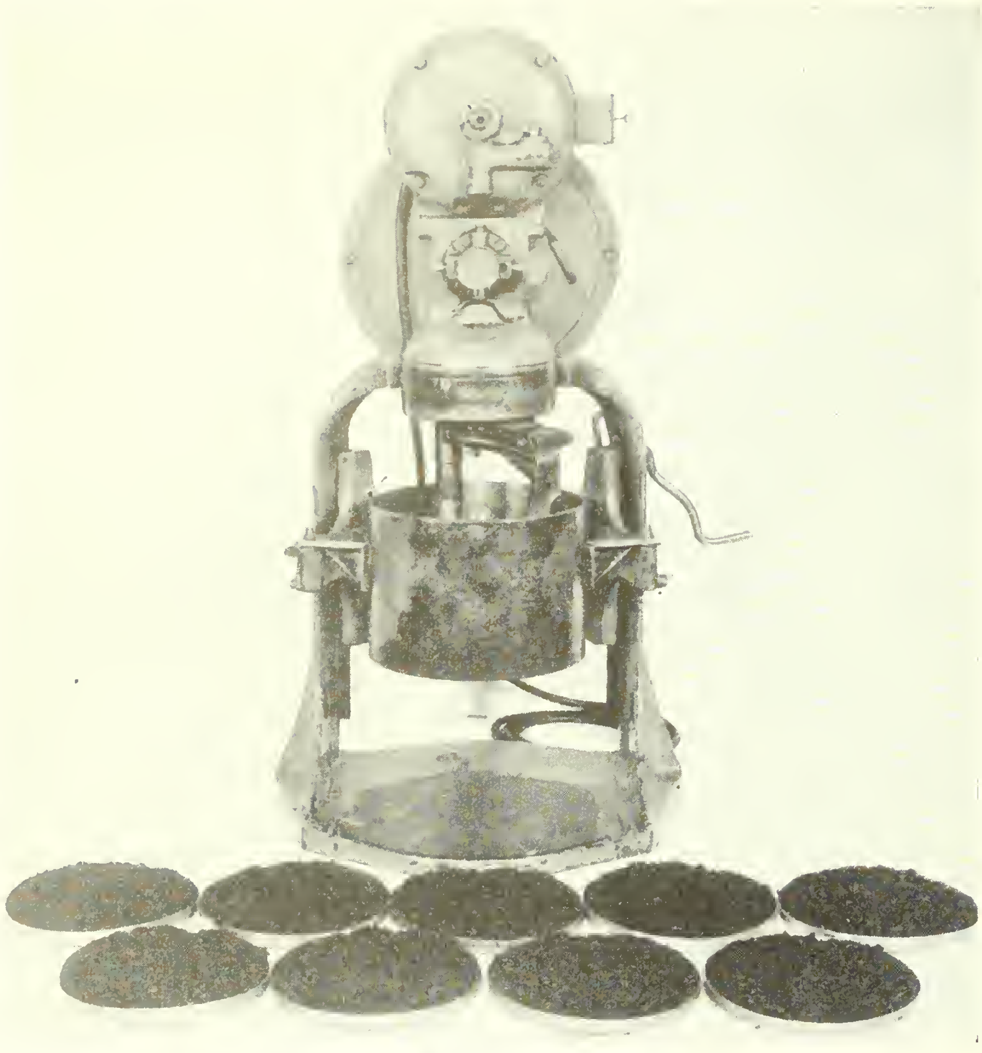

Figure 14.—Mixer and cooling pans.

subsequent cooling to air temperature, and the operations of weighing, heating, and molding are then carried out as described for the asphalt cements.

Molding and weathering

The test specimens are formed and tested in steel specimen cups having an inside diameter of 2 inches and a depth of ⅜ inch. A steel guide sleeve is provided to fit over the specimen cup during molding to retain the loose mixture and to center the compression plunger. Both the cup and the guide sleeve are warmed for 3 minutes in the 325°F. oven prior to being used in molding each test specimen.

The batch of exactly 30 grams of mixture, preheated for 9 minutes at 325°F., is quickly placed into the warm mold and compressed under a load of 1,000 pounds per square inch maintained for 1 minute. The actual molding temperature of the mixture will be approximately 300°F. if this procedure is followed carefully. After molding, the specimens contained in their steel cups are permitted to cool to approximately 77°F. prior to being weathered in the oven. Figure 15 shows the special equipment for molding.

Weathering periods of ¼, 1, 2, 4, and 8 hours are used. Three molded specimens are tested for each weathering period. Thus, 15 specimens are required for the full test. These are placed in an 18-inch square oven pan 3 inches in depth, covered with a 1/16-inch mesh screen, and exposed in an oven at 325°F. The pan and screen eliminate variable air draft and thus provide more uniform curing conditions for the specimens. After oven curing or weathering for the desired period at 325°F., and cooling to room temperature, the specimens together with the supporting cups are weighed and the weight is recorded. The specimens are then placed in the 77°F. air bath preparatory to being tested for resistance to abrasion.

Abrasion machine

Four thousand grams of lead shot, passing the No. 10 and retained on the No. 14 sieve, are used for each test unless a smaller amount causes sufficient erosion of the test specimen to expose the bottom of the retaining cup.

Figure 16 is a schematic drawing of the abrasion machine used in this study. It conforms essentially to the California design. The critical dimensions are shown on the drawing. The rate of rotation of the test specimen is approximately 340 r.p.m. The lead shot is dropped at a rate of approximately 2,000 grams per minute. The temperature of the test is controlled by placing the machine in an air bath in which the temperature is maintained at 77°F. Accurate temperature control is very important. The shot is stored in the air bath between tests.

Expressing abrasion loss

The abrasion loss is normally reported on the basis of loss per 1,000 grams of shot after a total of 4,000 grams of shot have been dropped. However, the test should be stopped short of 4,000 grams of shot in the event that the abrasion has progressed through the specimen to expose the bottom of the cup. In such cases, the loss per 1,000 grams is computed on the basis of the amount of shot dropped. The loss is determined to hundredths of a gram. Figure 15 shows specimens at various stages of abrasion.

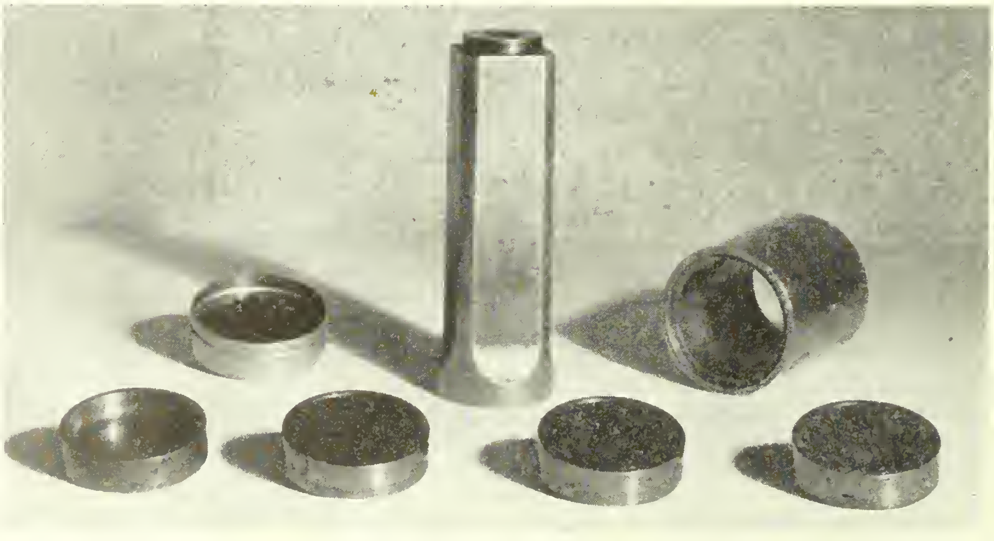

Figure 15.—In the background, special molding equipment. In the foreground, empty cup, freshly molded specimen, slightly abraded specimen, and highly abraded specimen.

WEATHERING-STRENGTH TEST PROCEDURE

Preparing the mixture

Two thousand grams of standard Ottawa sand (No. 20–No. 30) is required for each batch. Two batches are required for each set of 18 specimens. For asphalt-cement mixtures, the sand is heated to 325°F.; and for liquid asphaltic mixtures, the sand is at room temperature. Sufficient asphaltic material for each batch, while being heated to the mixing temperature, is stirred to prevent local overheating.

For mixing, the asphaltic materials are brought to the following temperatures:

Penetration grades (asphalt cement)	325°F.
RC or MC cutbacks	200°F.

Two thousand grams of sand and 80 grams, or 4 percent, of asphalt material are combined by mixing for about 2 minutes. The second batch to complete the full set of specimens is prepared in the same manner.

Curing the mixture

The prepared mixture, immediately after mixing, is spread to a uniform depth in pans 5.5 inches in diameter and ⅜ inch in depth, 200 grams of mix to each pan, and the pans set aside to cool to room temperature.

For the asphalt-cement mixtures, the pans containing the loose mix are then placed in the 325°F. oven for one-half hour. This is done at intervals of one every 5 minutes to allow time for molding. For the liquid asphaltic mixtures, a preliminary curing period of 18 hours in a 140°F. oven is used prior to the half-hour curing described above.

Molding test specimens

Molding follows immediately after the half-hour curing in the 325°F. oven. Molding is done by the double-plunger method with the molds preheated in a water bath to approximately 180°F. Enough of the 200 grams of mix (about 180 grams) from each pan is used to produce a cylinder 2 inches in diameter and 2 inches in height. It is important that the pans be removed from the oven in the same order in which they were put into the oven.

A molding pressure of 2,000 pounds per square inch is used and is held for 2 minutes. The temperature of the mix will be approximately 260°F. at the time of applying the molding load if the procedure is carefully followed.

After molding, the bottom plunger is removed, a base plate is inserted and the specimen pressed to the bottom of the mold

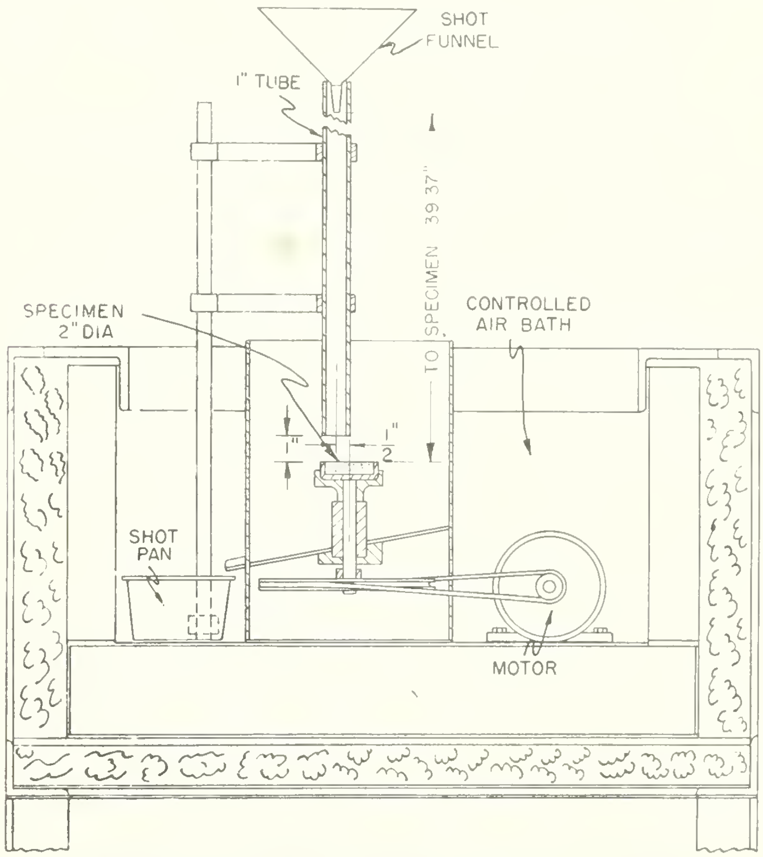

Figure 16.—Schematic drawing of abrasion machine.

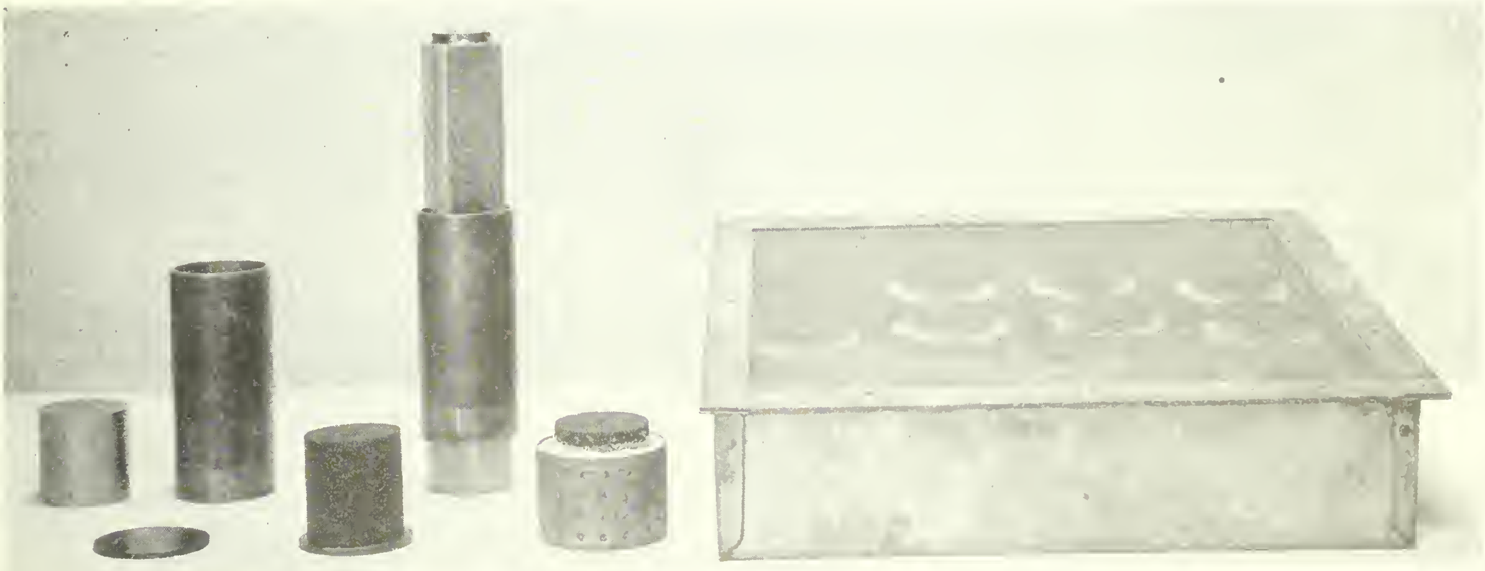

Figure 17.—Base plunger, base plate, centering and retaining sleeve, molded specimen, assembled mold, specimen supported by sand, and pan with screen cover used in oven weathering.

against the top of the plate. The molding cylinders containing the molded specimens are then placed in a 77°F. air bath for about 16 hours. After cooling, the specimens, with the base plates adhering, are pressed out of the mold and placed in specially prepared 6-ounce cans to undergo oven weathering.

The 6-ounce cans are perforated with sixty-four 1/16-inch holes on the side. As the specimens are placed in the perforated cans, Ottawa sand is filled in around the specimens in order to prevent their deforming during the period of heat treatment in the oven.

Oven exposure

The cans containing the specimens and reinforced with No. 20–No. 30 Ottawa sand, as described above, are then placed in an oven pan about 18 inches square and 3 inches in depth, covered with a 1/16-inch mesh screen, and exposed in an oven at 325°F. The pan and screen eliminate variable air drafts and thus provide more uniform curing conditions for the specimens.

Three specimens are required for each oven-weathering period. Weathering periods of 1, 2, 3, 4, 6, and 8 hours have been used. Each set of three containers with their respective specimens are removed at the end of each oven-exposure period and placed in the 77°F. air bath prior to being tested in compression. Figure 17 shows the apparatus used in this test procedure.

Compressive strength

After cooling to 77°F., the specimens are removed from the cans, the base plates removed, and adhering Ottawa sand and particles brushed off. They are then tested in compression at a unit deformation rate of 0.1 inch per minute. The compressive strength and the deformation at the point of maximum strength are recorded.

THIN-FILM OVEN TEST PROCEDURE

Scope

The thin-film oven test is intended for the determination of the effect of heat and air on asphaltic materials when heated in thin films as hereinafter prescribed. The loss in weight, and a comparison of the penetration, softening point, and ductility before and after heating are used as a measure of the effect of this test on asphaltic materials. For additional information of the changes that occur in the asphaltic materials, solubility and Oliensis tests also may be used.

While this test is prescribed primarily for asphalt cements, it may also be useful for cutback asphalts. For the latter material, the test should be made on the residue from distillation obtained by the standard method of test for cutback asphaltic products, A.S.T.M. Designation D 402.

Preparing the mixture

Samples of the bituminous material, 50.0 ± 0.5 milliliters in volume, are weighed (weight calculated from specific gravity) into a tarred aluminum pan 5.5 inches in diameter and ⅜-inch deep, with a flat bottom. A 50-milliliter sample in this size container gives a film thickness of approximately one-eighth inch. Semisolid materials should be heated to a fluid condition previously. If the material has been heated to facilitate transfer, the container and sample should be cooled to room temperature. The weight should be determined to the nearest hundredth of a gram.

Oven exposure

An oven conforming to the oven specified in the tentative method of test for loss on heating of oil and asphaltic compounds, A.S.T.M. Designation D 6, is used, except that the circular shelf is constructed so that it will support one or more of the sample containers in a horizontal position.

With the oven at 325°F., the sample in its container is placed on the circular shelf in the oven and the shelf is rotated at a rate of 5 to 6 r.p.m., for 5 hours. At the conclusion of the heating period, the sample is removed from the oven, cooled to room temperature, and weighed to the nearest hundredth of a gram. The loss due to heating is then calculated.

Tests of residue

The residue from the thin-film oven test is melted at the lowest possible temperature, mixed thoroughly, and transferred to a 3- or 6-ounce container as used for the A.S.T.M. penetration test. In order to provide sufficient material for tests on the residue, it is often necessary to make the thin-film oven test in duplicate, in which case the residues are combined for testing. The thoroughly mixed residue is tested according to A.S.T.M. standard methods of test for penetration (Designation D 5), softening point (Designation D 36), and ductility (Designation D 113).

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Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
Title Sheets for PUBLIC ROADS, volumes 24, 25, and 26.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1953

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$7,976	$23,520	$12,009	373.4	$8,104	$4,103	223.0	$34,674	$17,677	487.2	$66,298	$33,789	1,083.6
Arizona	803	5,520	3,864	119.1	1,906	1,337	13.8	6,088	4,027	88.0	13,514	9,228	220.9
Arkansas	7,552	8,344	4,493	256.9	4,253	2,144	185.8	14,589	7,422	287.0	27,186	14,059	729.7
California	3,647	14,481	6,667	101.9	20,747	10,272	60.3	101,775	50,758	283.1	137,003	67,697	445.3
Colorado	3,397	9,814	5,556	221.3	1,633	923	32.7	12,701	6,922	194.3	24,148	13,401	448.3
Connecticut	7,274	1,326	715	2.6	1,775	874	2.3	13,008	6,403	33.9	16,109	7,992	38.8
Delaware	2,820	1,668	836	8.4	55	28	--	7,120	3,602	31.9	8,843	4,466	40.3
Florida	4,565	21,435	11,048	329.7	5,860	3,018	124.3	15,038	7,587	216.4	42,333	21,653	670.4
Georgia	8,878	21,355	10,901	583.4	3,605	1,778	48.9	35,635	17,086	553.2	60,595	29,765	1,185.5
Idaho	3,011	12,111	7,356	312.6	3,543	2,212	72.8	10,245	6,523	189.5	25,899	16,091	574.9
Illinois	9,857	39,654	21,667	209.1	25,342	12,727	237.9	62,407	32,442	529.0	127,403	66,836	976.0
Indiana	10,525	37,435	19,549	170.1	11,165	5,520	87.0	27,677	14,882	189.2	76,277	39,951	446.3
Iowa	4,275	17,359	9,253	570.4	8,629	5,218	362.5	13,320	6,660	544.5	39,308	21,131	1,477.4
Kansas	5,526	11,810	5,829	956.4	9.330	4,557	488.5	14.413	6,969	642.6	35,553	17,355	2,087.5
Kentucky	4,332	14,122	7,338	194.8	7,283	4,019	96.7	19,931	10,046	306.8	41,336	21,403	598.3
Louisiana	4,108	16.713	8.385	124.1	8,869	4,424	58.4	23,005	11,017	121.3	48,587	23,826	303.8
Maine	1,163	9,996	5,003	86.3	442	231	.5	13,927	6,748	86.5	24,365	11,982	173.3
Maryland	7,954	10,005	5,176	79.8	2,952	1,338	26.4	8,647	4,779	38.9	21,604	11,293	145.1
Massachusetts	8,618	8,620	4,455	16.7	858	429	--	40,356	19,220	40.1	49,834	24,104	56.8
Michigan	8,163	29,418	15 248	585.6	7,991	4,027	188.6	55.558	24,105	237.1	92,967	43,380	1,011.3
Minnesota	4,875	12,078	6,611	943.6	10,769	5,600	784.6	16,491	8,943	484.8	39,338	21,154	2,213.0
Mississippi	2,083	17,106	8,617	584.6	5,085	2,523	235.9	20,481	10,537	556.6	42.672	21,677	1,377.1
Missouri	10,157	16,989	8,917	825.6	8,897	4,454	179.7	55,542	28,131	479.8	81,428	41,502	1,485.1
Montana	7,086	13,506	8,596	275.4	4,904	2,853	140.3	16,966	10,216	285.0	35,376	21,665	700.7
Nebraska	13,084	13,210	7,073	546.9	4,753	2,705	167.1	12,458	6,784	285.1	30,421	16,562	999.1
Nevada	5,166	4,260	3,565	73.8	883	739	35.6	6,261	4,875	178.2	11,404	9,179	287.6
New Hampshire	2,311	4,083	2,042	18.6	1,070	533	8.0	6,222	3,232	38.9	11,375	5,807	65.5
New Jersey	6,438	8,952	4,397	63.2	2,094	962	3.0	31,671	15,312	38.1	42,717	20,671	104.3
New Mexico	1,462	4,845	3,089	113.9	3,189	2,019	69.6	7,016	4,458	157.9	15,050	9,566	341.4
New York	24,133	91,698	48,018	199.9	40,597	20,531	147.1	126,376	58,545	345.9	258,671	127,094	692.9
North Carolina	6,740	22,781	11,033	453.2	4,570	2,232	189.6	32,026	15,425	597.0	59,377	28,690	1,239.8
North Dakota	2,445	5,344	2,686	953.1	7,888	3,944	699.9	9,028	4,691	656.5	22,260	11,321	2,309.5
Ohio	8,357	24,724	11,796	149.1	23,859	11,455	82.2	79,483	40,198	101.6	128,066	63,449	332.9
Oklahoma	11,917	8,332	4,605	186.7	6,002	3,162	92.8	16,309	8,571	239.8	30,643	16,338	519.3
Oregon	3,308	858	463	15.2	3,760	2,250	103.5	15,259	9,162	215.2	19,877	11,875	333.9
Pennsylvania	5,707	43,393	20,168	75.4	26,465	12,591	64.3	86,731	43,152	218.2	156,589	75,911	357.9
Rhode Island	2,008	3,277	1,638	31.9	161	80	--	19,316	10,037	28.9	22,754	11,755	60.8
South Carolina	5,936	7,881	4,371	178.4	2,704	1,331	142.2	19,478	10,026	377.9	30,063	15,728	698.5
South Dakota	1,363	9,938	5,902	678.6	4,225	2,410	240.7	8,198	4,940	392.1	22,361	13,252	1,311.4
Tennessee	4,301	16,733	9,254	537.4	7.657	3,672	209.3	35.338	15,780	283.0	59,728	28,706	1,029.7
Texas	17,144	4,714	2,380	103.1	18,940	10,756	380.9	57,060	31,068	900.1	80,714	44,204	1,384.1
Utah	490	3,536	2,698	52.4	2,758	2,088	57.3	12,706	9,594	174.9	19,000	14,380	284.6
Vermont	576	4,501	2,435	46.9	1,486	743	12.4	8,839	4,404	60.5	14,826	7,582	119.8
Virginia	2,153	16,102	6,901	171.8	9,655	4,561	142.2	33,278	16,380	268.8	59,035	27,842	582.8
Washington	2,538	9,633	4,954	193.5	4,606	2,536	91.1	14,522	7,689	109.7	28,761	15,179	394.3
West Virginia	5,289	6,847	3,454	43.2	3,960	2,004	6.2	19,700	9,863	157.4	30,507	15,321	206.8
Wisconsin	4,675	14,057	7.657	268.4	7,464	3,568	190.0	38,823	19,703	404.1	60,344	30,928	862.5
Wyoming	975	3,043	1,996	77.7	2,365	1,550	93.5	7,672	5,056	136.5	13,080	8,602	307.7
Hawaii	1,460	2,919	1,426	7.5	1,526	738	5.2	10,225	4,944	19.4	14,670	7,108	32.1
District of Columbia	1,176	7,808	3,664	6.0	4,837	2,214	2.0	11,148	5,282	1.9	23,793	11,160	9.9
Puerto Rico	3,673	11,496	5,322	57.3	1,174	572	4.8	13,899	6,633	44.7	26,569	12,527	106.8
TOTAL	283,470	729,350	381,076	13,234.9	362,645	186,555	6,891.4	1,378,636	698,506	13.339.0	2,470,631	1,266,137	33,465.3

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VOL. 27, NO. 10

OCTOBER 1953

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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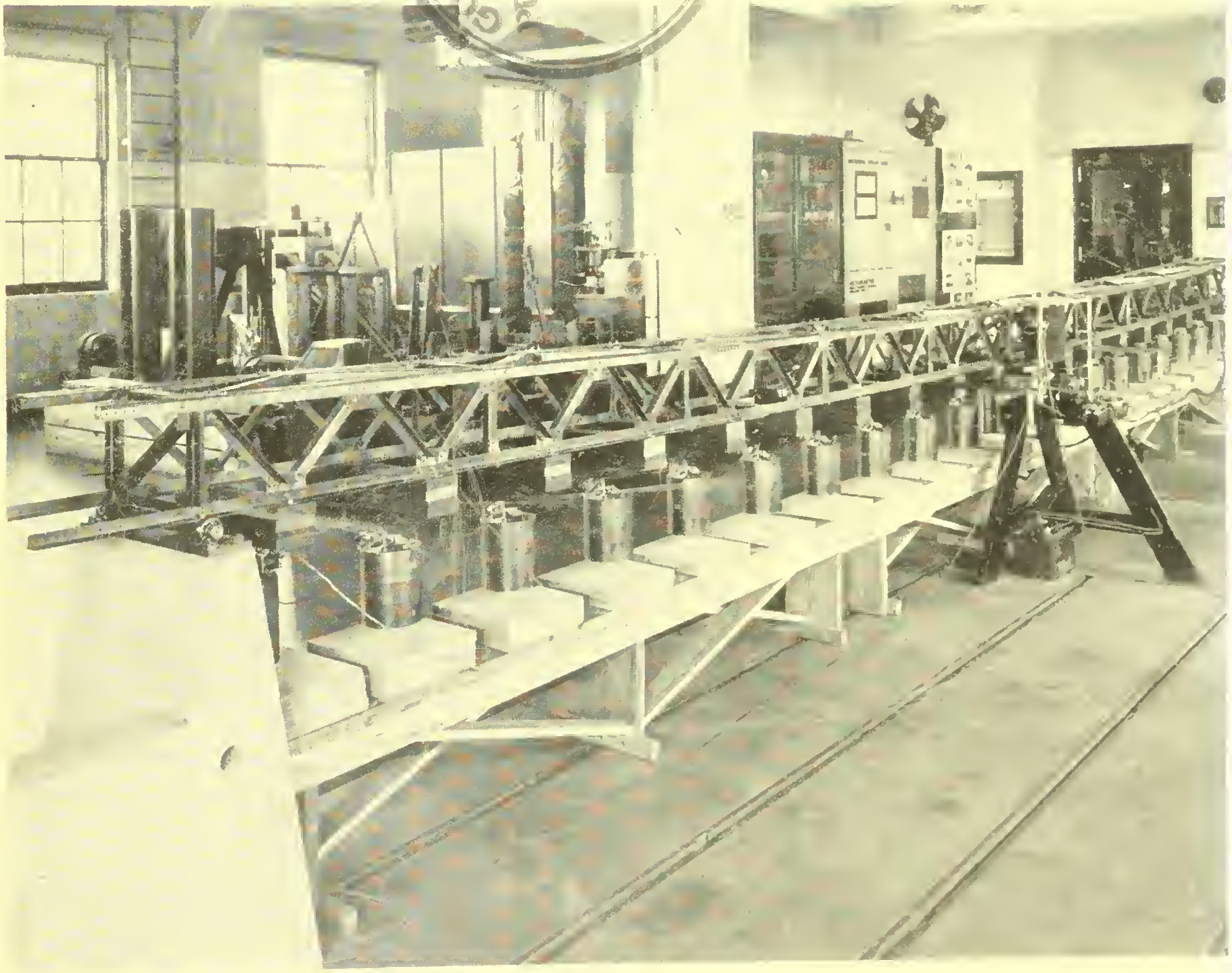

Bolted truss used in tests of structural damping

PUBLISHED BY
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WASHINGTON

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Published Bimonthly

Edgar A. Stromberg, *Editor*

BUREAU OF PUBLIC ROADS
Washington 25, D. C.

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The printing of this publication has been approved by the Director of the Bureau of the Budget, January 5, 1952.

Contents of this publication may be reprinted. Mention of source is requested.

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Results of Physical Tests of Road-Building Aggregate: a new publication 234

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
FRANCIS V. du PONT, Commissioner

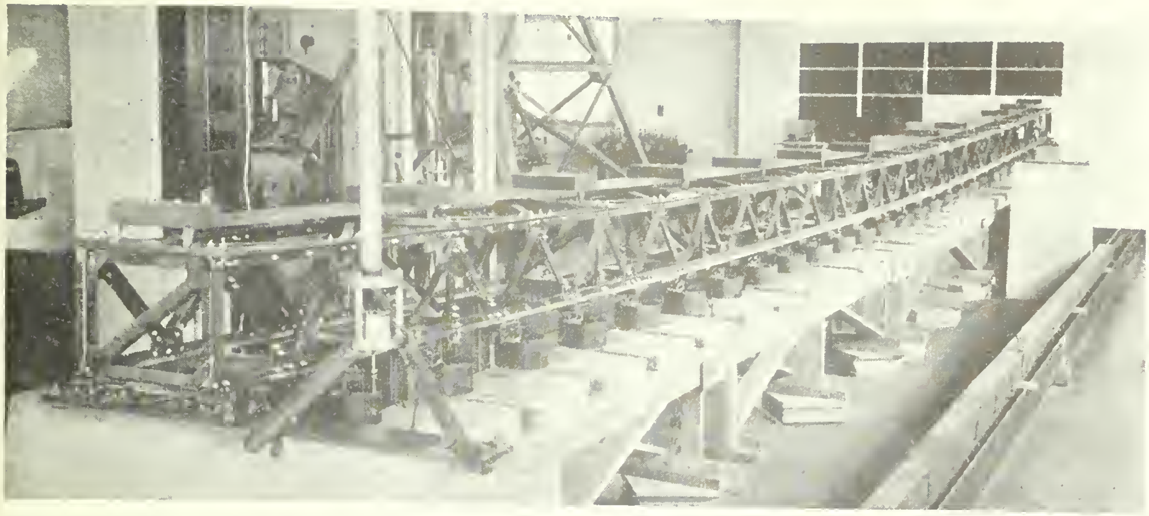

Figure 1.—General view of truss with simulated deck in place.

Tests of Structural Damping

Reported by LESLIE W. TELLER
Chief, Structural Research Section,
and ERNEST G. WILES
Highway Physical Research Engineer

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

In connection with a program of research sponsored by the Advisory Board on the Investigation of Suspension Bridges, information regarding the self-damping characteristics of steel structural members was needed. The literature contained little or no data on the subject. At the request of the Advisory Board, the Bureau of Public Roads conducted a series of tests which developed a considerable amount of useful data on the self-damping characteristics of both solid and trussed structural members together with data which show the possibilities of dry or Coulomb friction as a means for energy absorption within a framed structure. The details of the entire study are described in this report.

FOLLOWING the catastrophic failure of the suspension bridge across the Tacoma Narrows in the State of Washington, shortly after it was opened in 1940, there was organized, under the sponsorship of the Bureau of Public Roads, a technical body of broad representation known as the Advisory Board on the Investigation of Suspension Bridges. The objectives, organization, and membership of this body are well known and need not be repeated here.

At the request of the Advisory Board, as a part of its extensive investigational program, the studies of structural damping described in this report were made in the research laboratories of the Bureau of Public Roads. A condensed account of these studies has been published previously.[1] The present report is intended to describe the experimental work more fully than was possible in the A.S.C.E. paper and, further, to present certain additional data obtained as the program was concluded.

Since the failure of the Tacoma Narrows Bridge was caused by vibrations set up by wind action, the investigational work of the Advisory Board has included extensive tests of both full models and sectional models under the controlled conditions of a wind tunnel. Necessarily these models are dimensioned to a greatly reduced scale and it is not possible, as a practical matter, to reproduce in the model all of the structural details of the prototype. For this reason, in the fabrication of full models—that is, models of entire structures—it has been the practice to simulate the stiffness of the stiffening girders or stiffening trusses of the prototype by means of solid steel members and to simulate their shapes by other means in order that the aerodynamic action of the model would be similar to that of the prototype.

It is desirable that the energy which a suspension bridge receives from the wind be dissipated to the maximum extent possible by friction and by other means so that the energy remaining will not be able to cause excessive vibrations in the bridge. Friction within the structure offers one important means for absorbing energy received from without.

The Basic Question

Since movement at the joints of stiffening trusses may be an important source of friction, a question naturally arose as to whether or not, in the case of a full model, the substitution of a solid member for a trussed member might lead to significant differences in energy absorption and thus affect the validity of certain data obtained in the wind tunnel tests. The investigational work described in this report was undertaken initially to find an answer to this question. In addition the tests provided basic data useful in a theoretical approach to the problem of structural damping in suspension bridges [1]. The results obtained in the early work led to some expansion of the initial program in order that other useful information on structural damping might be obtained while the experimental facilities were available.

In general, damping is the dissipation of energy that a vibrating structure receives from external exciting forces, in the course of which part of the external energy is transformed into molecular energy and part is dissipated to surrounding objects or the atmosphere. The structural damping discussed in this report is the result of one or more of the following: imperfect elasticity of the material, friction due to small relative displacements in the joints of the structure, and friction at sliding expansion joints of a simulated floor system, at sliding end bearings, or at the sliding surfaces of mechanical brakes.

---

[1] *Structural damping in suspension bridges,* by Friedrich Bleich and L. W. Teller. Transactions, American Society of Civil Engineers, vol. 117 (1952), p. 165 (also printed as a separate, Paper No. 2486).

The program of damping tests was originally recommended by the Committee on Model Studies of the Advisory Board and was approved by the Executive Committee of the Board at a meeting held in New York on October 8, 1943. The results obtained were placed before the Board in a series of four progress reports dated March 1945, April 1946, May 1948, and July 1949, respectively. The data contained in these reports were released in condensed form in the A.S.C.E. paper previously referred to.[2]

To carry out the program, three test specimens were obtained through the cooperation of the American Institute of Steel Construction and the Inland Steel Company. These were a bolted truss, a rolled **H**-section, and a solid bar of rectangular cross section. The material of which the truss was made was a low alloy steel of relatively high strength, while that in the **H**-section and the rectangular bar was carbon steel with 0.20–0.25 percent carbon. The specimens were supported at the ends in a horizontal position on either a 36-foot, 10-inch, or a 28-foot, 4-inch span length, as will be described in detail later.

Scope of Investigation and Summary of Indications

The expanded program of the investigation authorized by the Advisory Board consisted of four principal parts, plus a number of subordinate collateral investigations. Some supplemental studies were also made to develop additional information. The nature of the investigations and summarized findings are described in the following paragraphs.

Principal studies

1. A comparison was made of the self-damping characteristics of a trussed structural member with that of solid structural members in equal stiffness, when vibrated in the first symmetric mode.

The character of the amplitude decay for the two solid sections tested was definitely exponential while that for the bolted truss was more complex, being apparently the result of a combination of the imperfect elasticity of the material with plastic yielding and friction due to small displacements in the joints. In these tests there was, of course, end bearing friction and air friction, but the former was present as a constant and the latter was believed to be relatively unimportant.

In comparisons between the solid members and the bolted truss, the truss showed considerably more capacity for self-damping, particularly at the higher amplitudes (or higher dynamic stresses). With the joints tightly bolted the decrement values ranged from about 0.006 at 1,000 p.s.i. to about 0.030 at 10,000 p.s.i. dynamic stress in comparison with a range of 0.003 to 0.007 for the **H**-section at the same limits of dynamic stress.

The decrement values for all three specimens increased with an increase in amplitude (or dynamic stress), the rate of increase being greatest with the truss and least with the **H**-section.

For small amplitudes and small dynamic stresses the decrement values were of the general order of 0.004 for all specimens. The decrement values for the solid members did not exceed 0.007 in any of the tests. For the truss, however, values as high as 0.073 were found at high initial amplitudes (and high dynamic stresses) in combination with low bolt tension in the bolted connections.

2. In the case of the bolted truss, a determination was made of the effect on damping of bolt type, bolt tension, and the procedure used in arriving at a given level of bolt tension.

The tightness of the bolted connections had a pronounced effect on the damping of the truss, particularly at the larger amplitudes (or dynamic stresses). The tighter the bolts, the lower were the decrement values.

The damping action of the joints in the truss when tightly bolted apparently was influenced more by the pressure of the surfaces held in contact by the bolts than it was by the fit of the bolts in the holes.

Tests with the bolted truss to study the effect of bolt tightening sequence on damping showed a measurable effect, it being least with the bolts tightened to the greatest degree of tension. The care taken in the bolt tightening procedure in all tests with the truss seems fully justified.

3. A study of the effects of a variation in dead load on the self-damping characteristics of the bolted truss showed that variation in the dead load of the order of 50 percent caused no measurable change in the damping characteristics of the truss.

4. A study was made of the effect of dry or Coulomb friction, externally applied, on the damping characteristics of the bolted truss. The Coulomb friction was developed in three different ways—by mechanical brakes, by sliding bearings at the ends of panels of a simulated floor system, and by sliding end bearings.

The external application of dry or Coulomb friction greatly increased the damping of the bolted truss.

Where mechanical brakes that employed metal plates rubbing together without lubrication were used to develop frictional forces for damping purposes, more dependable action was obtained with opposing surfaces of bronze and steel than with two ferrous metals in contact.

The damping obtained from the rubbing of steel on bronze without lubrication follows the laws of Coulomb friction—that is, it varies directly with the total normal pressure on the friction surfaces, is independent of their area, and is nearly independent of velocity. The effective coefficient for the condition of this test was about 0.13–0.14.

The increase in the damping of the bolted truss obtained by the application of the simulated deck sections was quite similar in character to that obtained with mechanical brakes in operation at the ends of the truss.

The application of the simulated deck to the bolted truss and the frictional forces developed at the sliding shoes of the deck sections added slightly but measurably to the stiffness of the truss during its dynamic oscillation. The effect was somewhat greater at small amplitudes than at large amplitudes of motion.

The substitution of sliding end bearings of bronze on cold rolled steel for the wheel bearings used at one end of the truss developed a strong damping force.

Collateral studies

The collateral investigations authorized by the Advisory Board were necessary to develop control data for the main investigation and for other purposes. Among the more important were:

(*a*) Determination of the ultimate strength and character of failure of a chord splice in tension and of a chord section in compression, as used in the bolted truss.

(*b*) Establishment of relations between load, deflection, and critical strain in each of the three specimens.

(*c*) Determination of the maximum tightening torque for the bolt-nut combinations used in the truss.

(*d*) Segregation of air damping effect when the truss was equipped with a simulated floor system.

The application of the plywood plates to the truss, in lieu of the steel deck panels of equal frontal area, increased the damping to a degree that could be measured, although the effect of the added air resistance on damping was relatively small.

(*e*) Determination of the coefficient of friction for the sliding bearings used.

(*f*) Determination of the chemical composition and physical properties of the steel in each of the three specimens.

Supplemental studies

After the work authorized by the Advisory Board had been completed, some supplemental studies were made which developed additional information of interest and value.

A. The damping characteristics of the truss with riveted connections were compared to its performance with bolted connections.

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[2] See reference, footnote 1, p. 203.

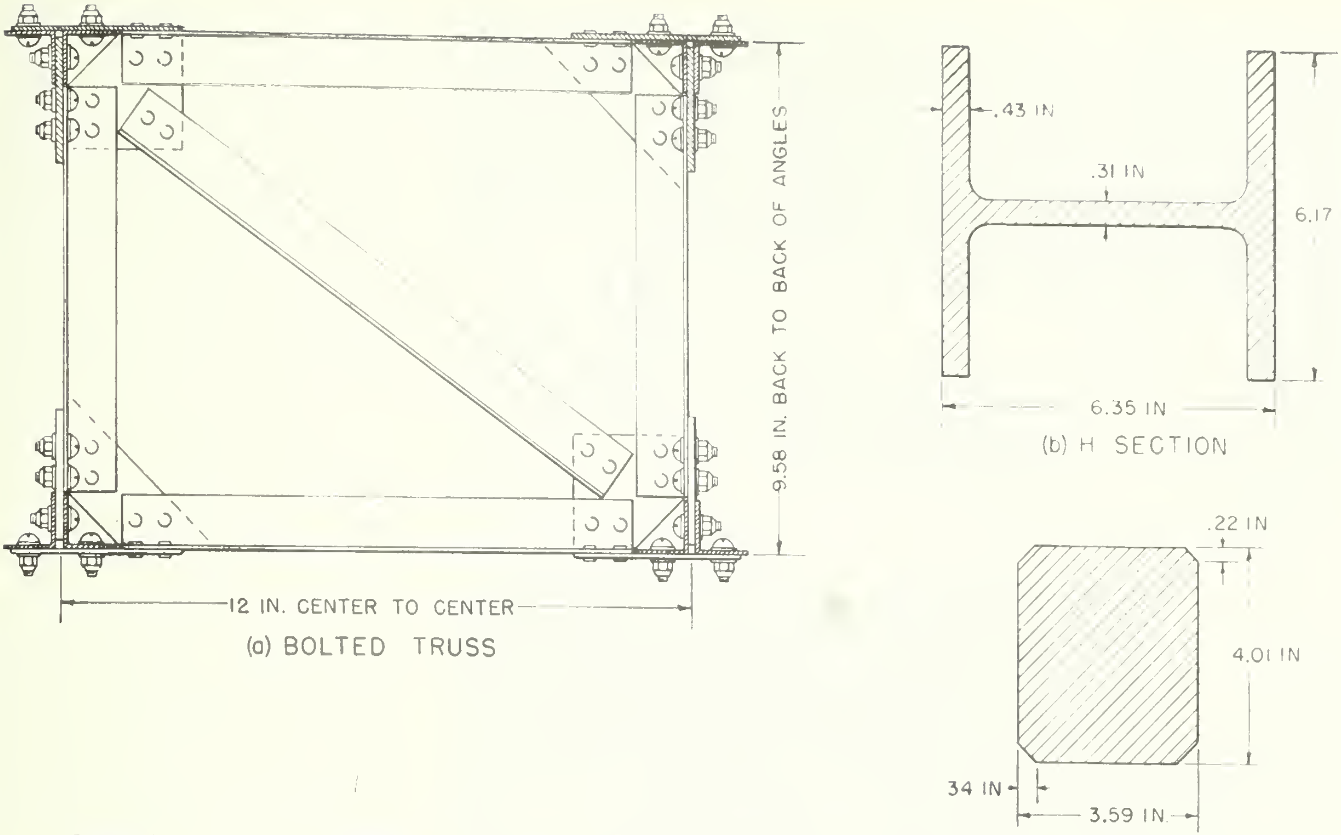

Figure 2.—Cross-section dimensions of the three specimens tested.

(a) BOLTED TRUSS

(b) H SECTION

(c) SOLID RECTANGULAR BAR

Tests with the truss with all chord connections riveted showed considerably less evidence of movement at the connections than did the tests with the bolted truss, particularly at higher amplitudes and higher dynamic stresses. Much greater damping was evident at all amplitudes than was found for the comparable solid member (**H**-section).

B. From comparison of logarithmic decrement values determined from strain decay data with those obtained from deflection decay data, it is indicated that, with suitable instrumentation, damping can be satisfactorily determined from continuous records of strain decay in tests of this type.

C. Measurements of rotation and translation at the ends of the **H**-section were found to be in good agreement with theory.

D. In comparisons of the theoretical and observed performance of the bolted truss it was found that a somewhat better correlation was obtained when a correction for the initial sag of the trussed member between end supports was introduced into the computations.[3]

The Test Specimens

While a search of the literature disclosed a certain amount of data on the damping characteristics of complete structures, particularly bridges of various span lengths, and data from a number of studies of the self-damping properties of steels of various types by means of laboratory tests on small specimens, there seemed to be no information on the self-damping characteristics of structural members made of steel. For this reason the investigation is believed to be unique. The test methods and instrumentation used in the tests were unusual in a number of respects and may be of general interest to the profession, so they are described fully in this report.

Table 1.—Comparison of physical characteristics of specimens

	36′–10″ span length				28′–4″ span length			
	Truss[1]		H–section		Truss (shortened)		Rectangular bar	
	Planned	Tested	Planned	Tested	Planned	Tested	Planned	Tested
Moment of inertia:								
From cross section.......in.^{4}..	19.1	20.3	19.1	16.9	19 1	20.3	19.1	19.0
From load deflection.......in.^{4}..	....	20.7	....	16.6	....	20.4	....	18.9
Weight per foot:								
Specimen proper.........lb..	6.72	7.26	27.50	24.00	6.72	7.59	48.70	48.26
Attached dead-load weights......lb..	20.78	21.13	0	0.16	41.98	41.96	0	0.36
Total dead weight..............lb..	27.50	28.39	27.50	24.16	48.70	49.55	48.70	48.62
Maximum deflection from dead weight.........................in..	2.06	[2] 2.05	2.06	2.05	1.28	[3] 1.24	1.28	1.27
Natural period..................sec..	0.41	0.39	0.41	0.41	0.32	0.31	0.32	0.32

[1] The following comparable values were obtained for the truss with all chord connections riveted: Moment of inertia from load deflection, 20.6 in.^{4}; maximum deflection from dead weight, 1.90 in.; natural period, 0.39 sec.

[2] Permanent set of 1.97 in. developed during testing with machine screws. Total deflection at end of test 4.02 in.

[3] Permanent set of 0.66 in. developed during testing with machine screws. Total deflection at end of test 1.90 in.

Table 2.—Physical properties of the steel in each specimen

Source and number of specimens	Ultimate strength	Yield strength (0.10 percent offset)	Modulus of elasticity	Reduction in area	Elongation (2 inches)	Rockwell hardness
	P.s.i.	*P.s.i.*	*P.s.i.*	*Percent*	*Percent*	
Truss (4)..............................	73,700	55,600	30,664,000	65	32	B 83
H-section (4)..............................	61,800	38,800	30,377,000	64	40	B 68
Bar[1] (2)..............................	57,000	25,000	30,590,000	19	18	B 61

[1] The material in the bar does not meet the tensile strength requirements of A.S.T.M. Designation A7–49T, specification for steel for bridges and buildings. This should have no significance insofar as the data obtained in these tests are concerned since the maximum stresses in the bar as tested were approximately 11,000 p.s.i.

As mentioned previously, the specimens furnished for the test program were a bolted truss, a rolled **H**-section, and a solid rectangular bar, all with cross-sectional dimensions as shown in figure 2. These were originally designed to provide sections of equal stiffness—that is, equal moments

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[3] See footnote 1, p. 203.

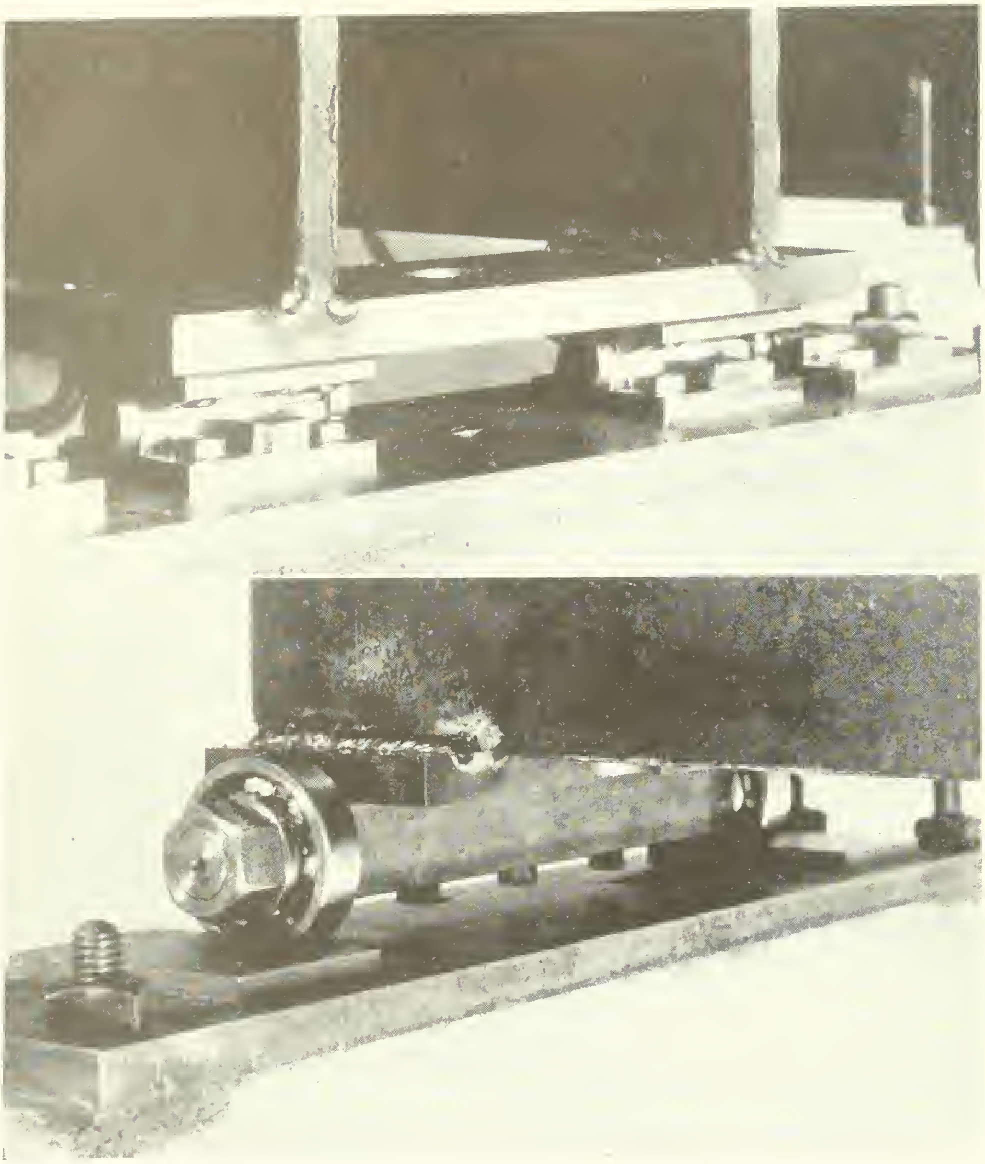


Figure 3.—Rocker bearing (above) and wheel bearing (below) used to support the specimens. The H-section is shown here.

of inertia. As furnished, however, there was some departure from the planned dimensions and a corresponding variation was found in the moment of inertia. The moment of inertia of the truss was slightly greater while that of the **H**-section was somewhat smaller than the moment of inertia of the rectangular bar.

The truss and the **H**-section were compared on a span length of 36 feet, 10 inches, and the truss (shortened) and the rectangular bar were compared on a span length of 28 feet, 4 inches.

In these comparisons, it was originally planned that the specimens being compared would be of equal weight and stiffness so as to have equal periods of vibration and equal dead-load deflections. For various reasons there were some deviations in the weights, dimensions, and stiffness of the test specimens as furnished, as may be noted in data which follow. There is no evidence, however, that the deviations mentioned were of sufficient magnitude to affect adversely the comparisons which have been made.

Since the weight of the truss as furnished was of the order of 250 pounds in contrast to 900 pounds for the **H**-section and 1,400 pounds for the bar, it was necessary to add a considerable amount of weight to the truss for the comparative tests. This weight was in the form of rectangular steel bars attached transversely at the lower panel points. These attached panel-point weights will be referred to as dead-load weights.

Certain pertinent information as to the weight, stiffness, and other physical characteristics of each specimen are given in table 1. Certain data on the physical properties of the material in each specimen are given in table 2.

Test Procedure

The general method of test was quite simple. The specimen was supported as a simple beam at the span lengths mentioned. It was excited to a given degree of oscillation in the first symmetric mode in the vertical plane. Having developed the desired amplitude, the oscillations were allowed to die out naturally through damping caused by internal friction, end bearing friction, and air friction. Since end bearing friction may be assumed to be constant for specimens of equal weight and since air friction could, at best, exert only a very small damping force under the conditions of these tests, it was felt that any important differences found in the data obtained with the various specimens would be caused by inherent differences in the damping forces within the specimens.

In a part of the program the damping effect from the friction sources listed above was augmented by controlled dry friction applied to the truss.

The rate of energy dissipation or damping can be determined from the way in which the oscillations of the specimen die out or decay, and can be measured by means of a quantity known as the logarithmic decrement of damping. This quantity may be defined as the natural logarithm of the ratio of the amplitudes of any two successive oscillations. If the rate of amplitude decay is exponential in character the logarithmic decrement will have a constant value; if not, the value of the decrement will be a variable. For convenience, in these tests, single amplitudes—that is, movements to one side of the mean position—have been used for decrement determinations. The method of analysis will be described later.

To support the specimen, substantial concrete pedestals were built. To the top of each pedestal a steel plate was attached after being carefully leveled. At one end of the span, rocker bearings were fastened to the plate to permit rotational movement of the specimen in the vertical plane, but to restrain it from any translatory motion. At the opposite end of the span the plate was used to provide a plane surface on which ball-bearing wheels attached to the specimen could roll. Details of these end bearings may be seen in figure 3.

Control of Oscillations

The means used to develop controlled oscillations in the specimens will be understood by referring to figure 4. Along the specimens, at intervals equal to the panel length of the truss, weights were suspended in such a manner that all could be released simultaneously. These weights were cylindrical in form, made of cast iron, and weighed 19.5, 34.5, or 51.75 pounds each, depending upon the test condition for which used. To distinguish them from dead-load weights that were attached to the specimens for various purposes, these suspended weights will be referred to as live-load weights in this report.

The live-load weights were supported by steel hooks extending downward from the specimen. Each hook terminated in a short horizontal knife edge. On this knife edge rested a weight support consisting of a pivoted trigger held in place by a latch which could be tripped by a solenoid, the entire mechanism being mounted on the suspended weight. By connecting the solenoids in series all of the weights could be released simultaneously by closing one switch.

In order to determine whether or not the pivoted trigger interfered in any way with the free fall of the suspended weight, tests were made in which the free fall was compared to that of an identical weight supported on a wire and released by melting the wire. Space–time traces of the two falling weights were found to be identical.

In most of the tests on the 36-foot, 10-inch span length 26 live-load weights were used, while on the 28-foot, 4-inch span length the number was 20.

A wooden bench and pads of celotex were provided to catch the released weights as the tests were made (fig. 4).

With this means of excitation it was possible to duplicate any desired test condition as many times as the program required.

Measurement of amplitude

In the majority of the tests the amplitude of the oscillation was recorded at midspan by means of a sharply pointed steel stylus, attached to the specimen in a position such that it could be brought into light contact with a smoked paper wrapped around a slowly revolving drum. By means of a solenoid under automatic control, the stylus traced on the smoked paper complete cycles of the vertical motion of the oscillating specimen. Usually these records were made for the initial oscillation and those that occurred during the first second of each 10-second interval thereafter.

During the intervening 9 seconds the stylus was retracted from contact with the paper in order to reduce to a minimum the damping from stylus friction. That this was accomplished to a satisfactory degree was indicated by tests which showed that the **H**-section vibrated perceptibly for about 13 minutes after the release of the live-load weights with no stylus contacts and for about 12 minutes with stylus contacts at 10-second intervals, as described above.

A time trace with 1-second intervals was also placed on the record by another stylus controlled by a calibrated pendulum.

The details of this deflection-recording equipment are shown in figure 5, and figure 6 is a photograph of a record (in negative form) as taken from the drum and laid flat. The drum frequently made more than one complete revolution during the course of a test, as it did in the case of the test record shown in figure 6. The traces of greatest amplitudes, recorded at 10-second intervals, were those made during the first revolution of the drum. Those with the next smaller amplitudes were made during the second revolution of the drum, and so on. The 1-second time-interval trace appears at the top.

In some of the later tests, where the damping rate was high and the intermittent recording could not be used effectively, a different method was used. In this a magnetic-reluctance pickup unit was employed with a galvanometer type oscillograph for recording. A 2,000-cycle carrier current was used in the system. Because of its limited displacement range the pickup unit was located near the rocker-bearing end of the truss, and the proportionality of vertical movement at this point with that at midspan was established by a comparison of simultaneous recordings. With this system there was no physical contact between the recording system and the moving specimen.

Strain gages used

Strain gages were placed on each specimen at midspan to provide data on the maximum stresses developed for the various conditions of test. These were of the electrical resistance type, known commercially as SR-4 gages, cemented to the specimen and used in conjunction with a static strain indicator or an oscillograph. One of the

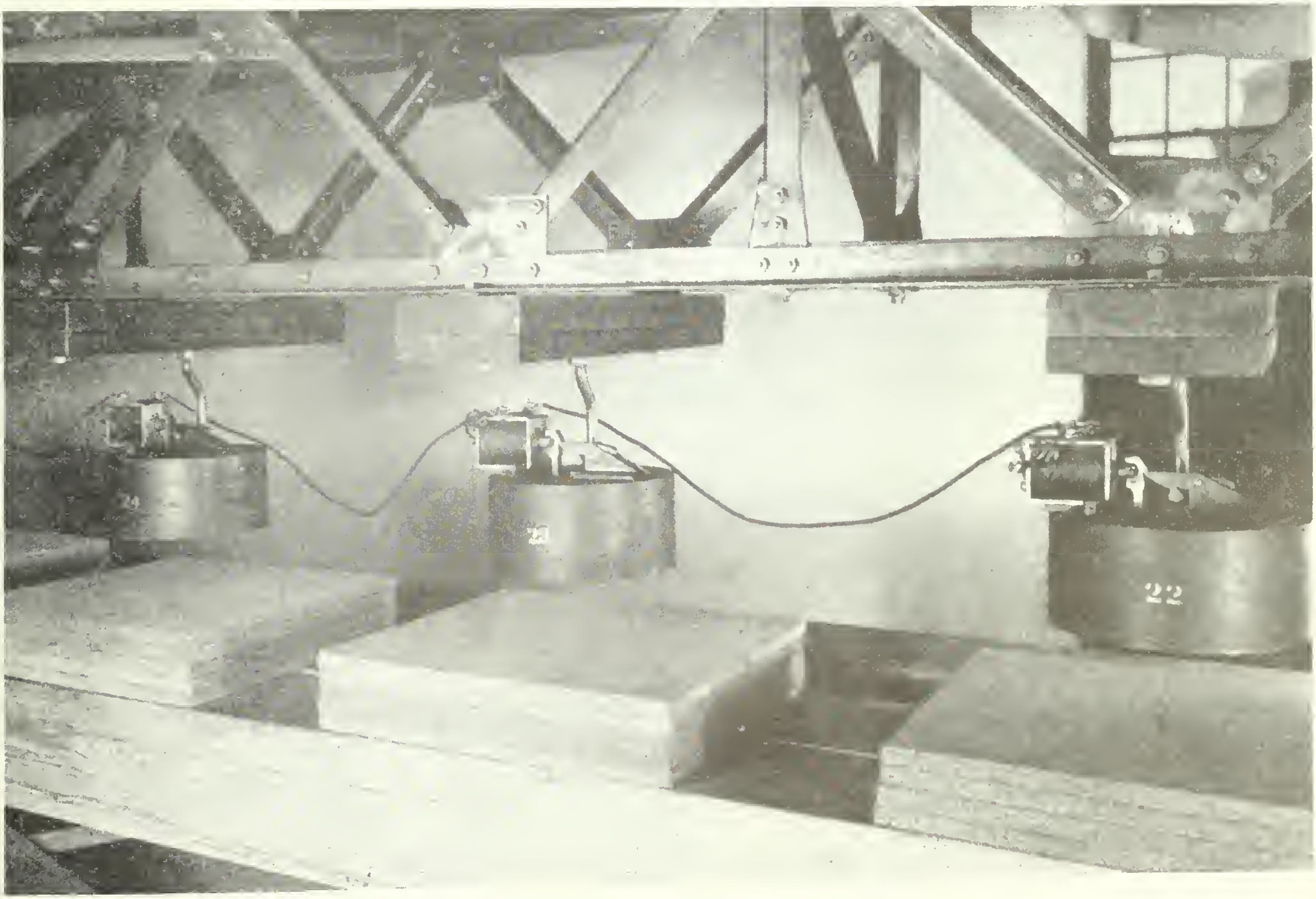


Figure 4.—Details of the weight-release mechanism.

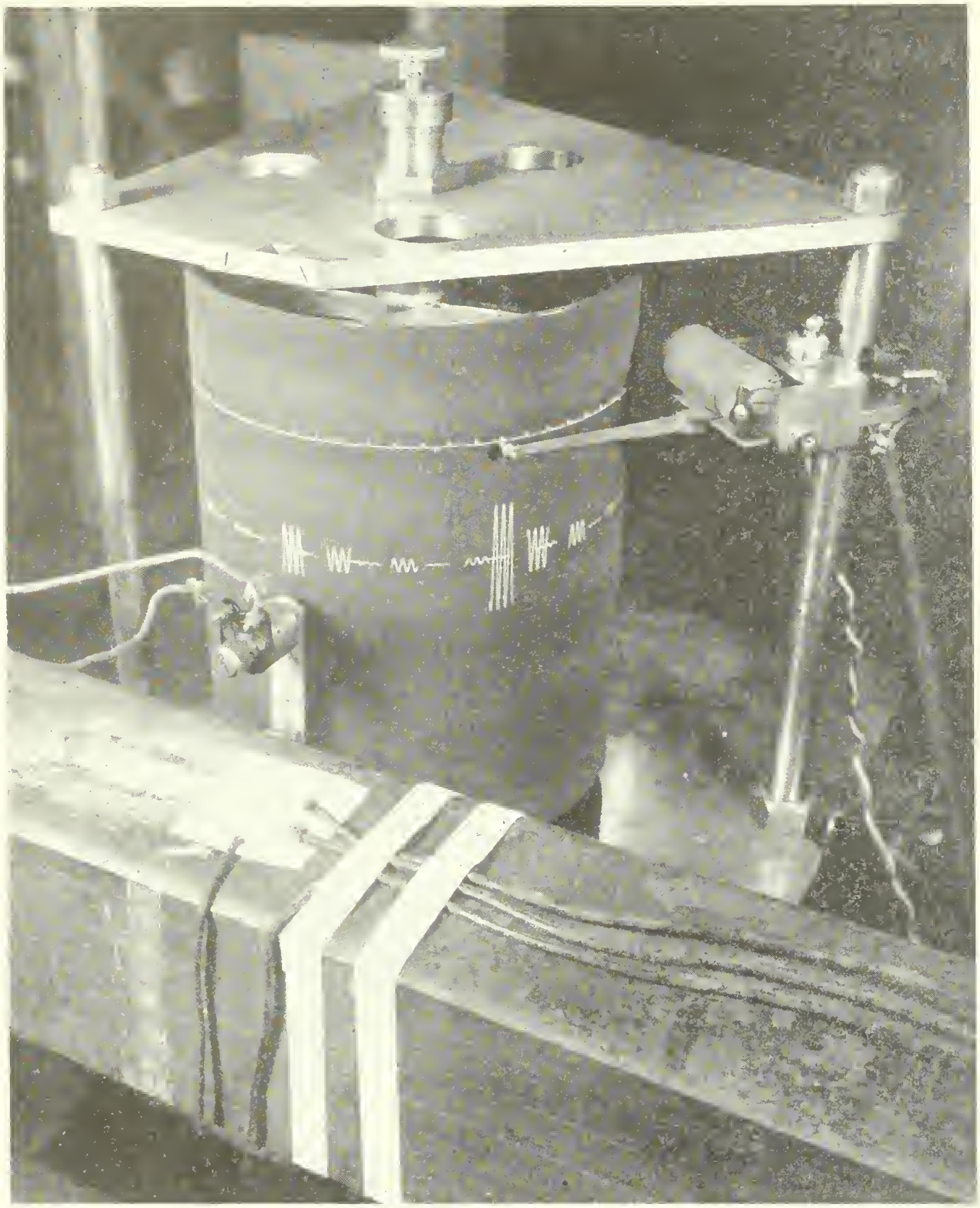


Figure 5.—The deflection-recording equipment: the solenoid-actuated styus at the left records vertical motion on the drum, while the stylus at the right records time traces. The bar specimen, with strain gage installed, appears below.

gages on the surface of the rectangular bar may be seen in figure 5 and typical strain data obtained on the **H**-section are shown in figure 7.

The relations between strain and load and between deflection and load were utilized for a number of purposes during the course of the investigation.

Some attempt was made in the early tests to determine decrement values from the rate of strain decay as recorded by an oscillograph. It was concluded that the method was feasible, but that the direct recording of deflection was perhaps a more reliable means since no electrical amplification was involved. However, in the supplemental investigation, with better instrumentation available, this question was studied more thoroughly with results that will be discussed later in the report.

The cover illustration shows the shortened truss as set up on the 28-foot, 4-inch span length for comparisons with the solid rectangular bar. The deflection measuring equipment, ball-bearing wheels at the (near) end frame of the truss, dead-load weights at the panel points, and the suspended live-load weights are to be seen in this photograph.

Chord Joints Tested

The trussed member was made up of angles, 1-inch by 1-inch by 0.065-inch in section, cold formed from a low alloy steel. Except for the lateral bracing in the plane of the upper chords, the joints of the truss as originally fabricated were drawn up with No. 10–24 commercial machine screws used as bolts. These passed through clearance holes in the gusset plates and truss members and were fitted with washers and hexagonal nuts of commercial grade. This was the type of joint fastening used in the initial tests.

A number of tests with a torque-indicating wrench showed that this screw-nut combination would withstand a tightening torque of 40 inch-pounds or more before failure occurred. On the basis of this information it was decided to test the trussed member (or truss, as it will be called) at three degrees of bolt tension, measured by tightening torques of 35, 25, and 15 inch-pounds, respectively.

The net section of a No. 10–24 machine screw is approximately one-half that of a solid rivet of the same overall diameter. This fact raised a question as to the strength of the chord splices in tension and it was decided to test such a splice to failure before beginning the program of

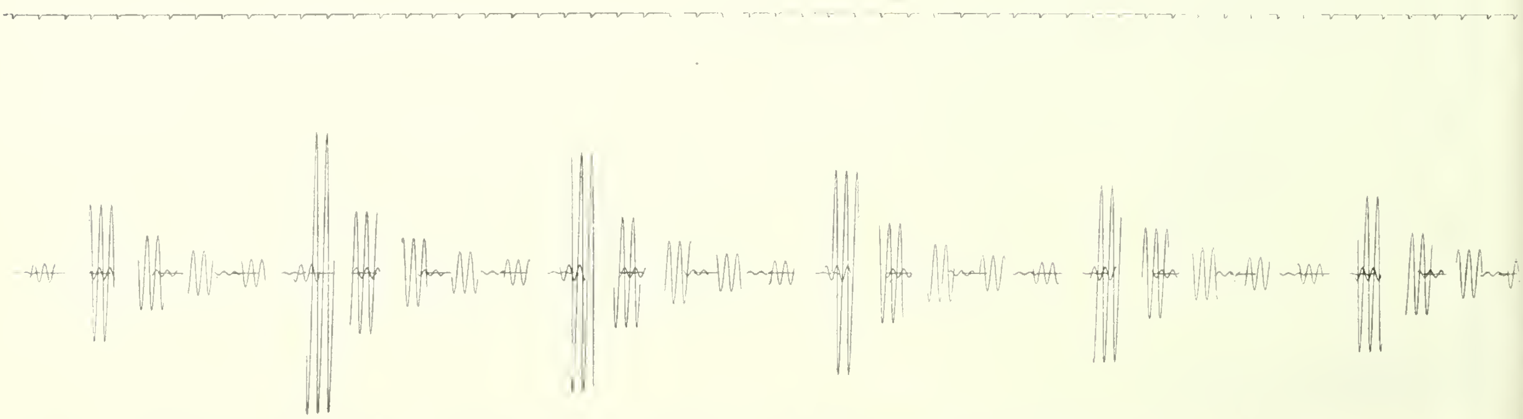


Figure 6.—Deflection record taken from the drum and laid flat. The oscillations persist during several revolutions of the drum.

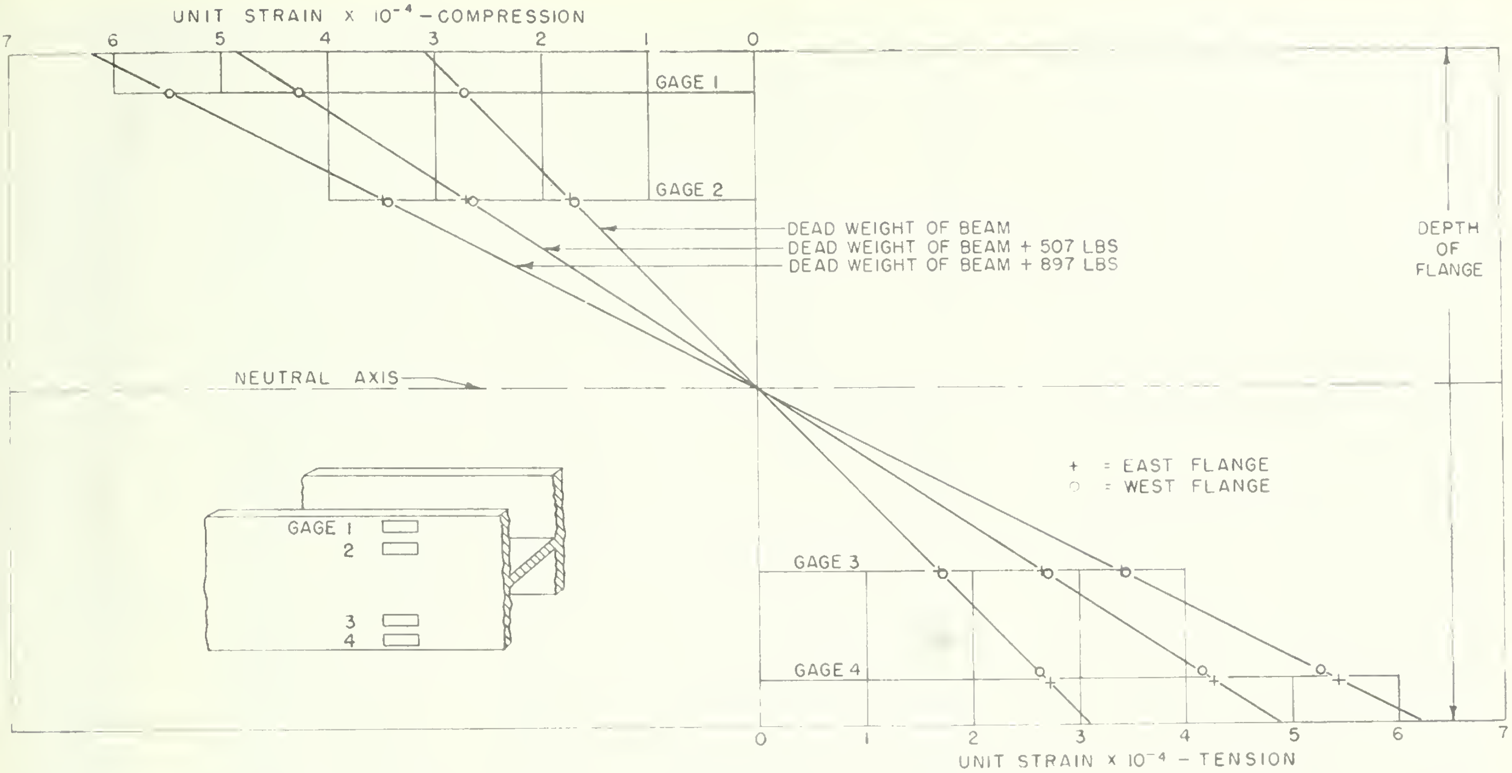


Figure 7.—Typical measured strain data on vertical faces of the H-section at midspan under various uniformly distributed loads.

load tests with the truss. The Inland Steel Company had on hand a few pieces of the angle from which the truss was fabricated and made these available. There was more than enough material for the one chord splice specimen desired for the tension test so as a matter of interest 34-inch specimens, representing a section of the upper chord between panel points, were made up and tested as columns in compression. Figure 8 shows the chord splice as set up in the testing machine for the tension test and figure 9 shows the set-up for the compression test. Restraint from bending at the ends of the compression specimen and restraint from deflecting laterally in one plane at the midpoint were provided to simulate the conditions under which the member would function. The bolts in the connections of both the tension and compression test specimens were tightened with a torque of 35 inch-pounds. From these tests the data contained in table 3 were obtained.

The tests of the chord sections in both tension and compression were considered important because of the rather high unit stresses expected in the chords under the heavier loadings (approximately 24,000 p.s.i.). On the basis of the results of these tests, as shown by the data in table 3, it was concluded that the truss would not be overstressed under any of the scheduled loadings.

Damping Characteristics of Truss and Solid Members

The truss bolted up with the No. 10–24 machine screws tightened to a value of 35 inch-pounds with a torque indicating wrench, as shown in figure 10, was compared with the **H**-section on the 36-foot, 10-inch span, the excitation being from the release of either the 19.5-pound or the 34.5-pound live-load weights. Similarly the shortened truss was compared with the rectangular bar on the 28-foot, 4-inch span, the excitation in this case being from the release of the 19.5-pound, 34.5-pound, or 51.75-pound live-load weights. Pertinent data on loads, deflections, and static-load stresses obtained in these tests are given in table 4.

From the data in this table it will be noted that under the various test conditions the initial double amplitudes at midspan varied from about 0.7 inch to about 4.1 inches and that the maximum stress in the extreme fiber at midspan ranged from about 6,350 p.s.i. to about 23,450 p.s.i.

As may be deduced from the record of oscillation decay shown in figure 6, the amplitude values decrease progressively with time as part of the energy is lost with each successive oscillation. If the energy loss is caused largely by viscous friction within the metal, as in the case of the **H**-section and the rectangular bar, the rate of decrease tends to be according to some exponential function. If, on the other hand, it is caused by external dry friction, the rate of decrease would tend to be linear. In these tests energy loss from both types of friction was present in varying degree, depending upon the particular conditions of test, as will be seen in data that will be presented.

Amplitude decay curve

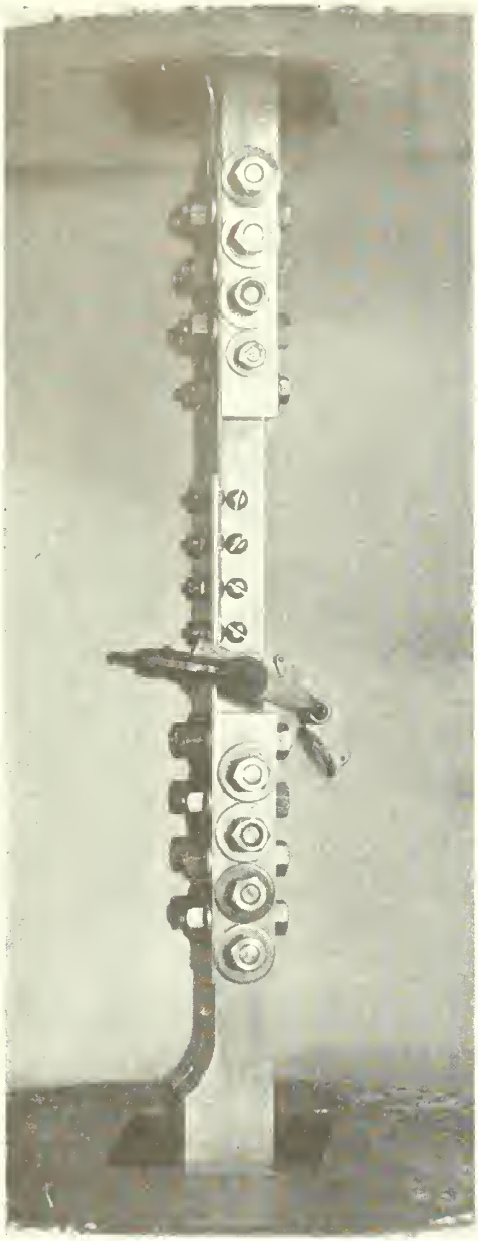
If a continuous record is made of the vertical movements of a point on the vibrating specimen and is related to a uniform horizontal time scale, and if a boundary or envelope curve is drawn through the extreme tips of the individual amplitude traces, a smooth curve will result. The shape of this envelope or amplitude decay curve is indicative of the type of friction that is causing the amplitudes to decrease, and from it values of the logarithmic decrement may be obtained. It will tend to be an exponential curve if viscous friction is the dominant cause of damping or tend toward

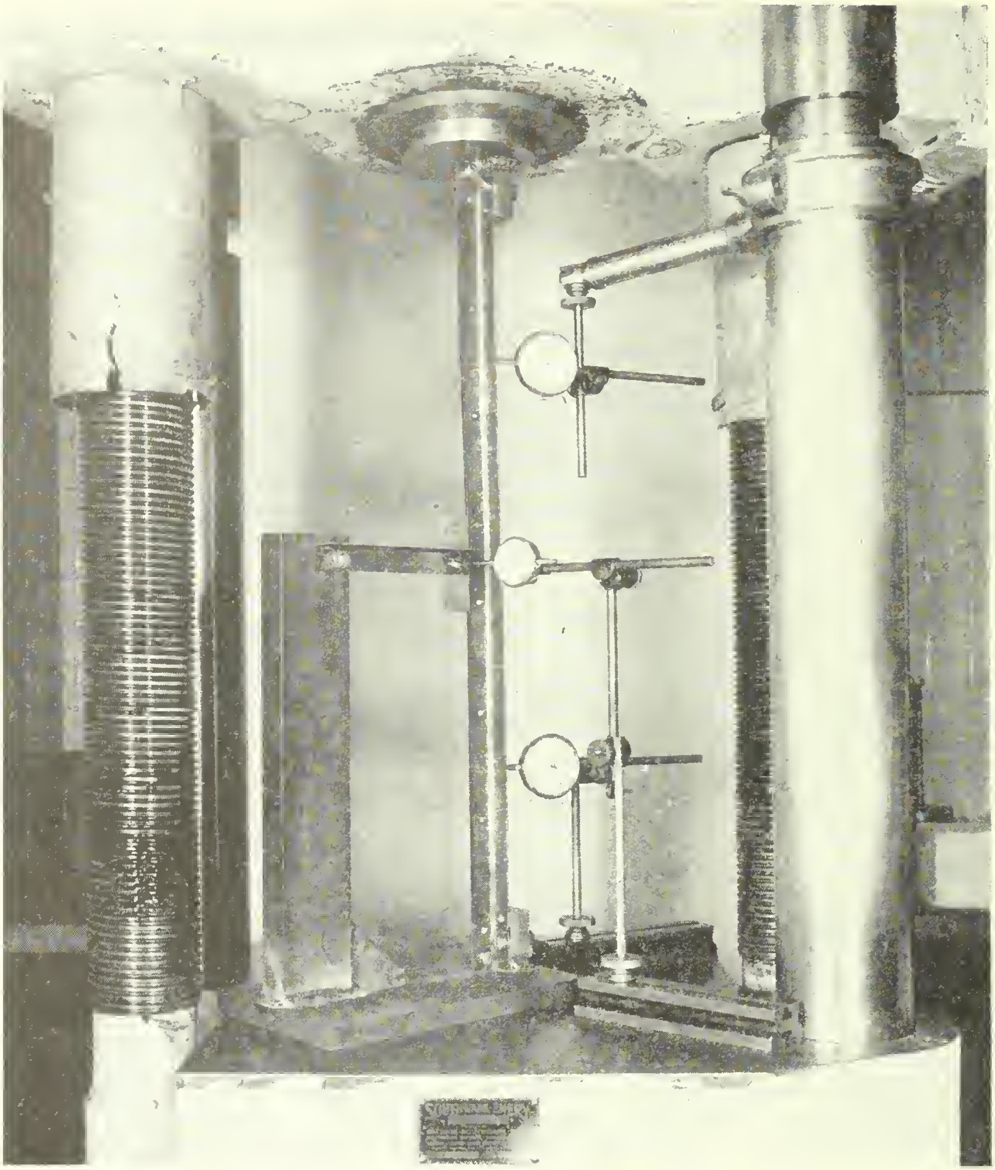
Table 3.—Tests of chord splices

	Fastenings		Load at failure	Stress in angles at failure	Type of failure
	Type	Torque applied			
		In.-lb.	*Lb.*	*P.s.i.*	
Tension tests: [1]					
Bolted splice	No. 10–24 machine screws	35	11,100	55,800	Bolts failed in shear
Bolted splice	NAS screws	15	15,220	76,600	Angles failed at bolt holes.
Riveted splice	Rivets	....	13,540	68,100	Rupture in splice plates.
Compression test [2]	No. 10–24 machine screws.	35	10,700	43,300	Buckling of angles at midlength.

[1] Net section of chord angles, 0.199 sq. in.

[2] Full section of two chord angles, 0.247 sq. in.





(LEFT) *Figure 8.—Tension test on chord splice of bolted truss.*

(ABOVE) *Figure 9.—Compression test on section of upper chord of bolted truss.*

Table 4.—Loads, initial amplitudes, and static load stresses

Member	Span length	Added weights		Dead load		Live load		Total load		Initial double amplitude		Stress in extreme fiber at midspan			
													Computed stresses		Tested
		Number	Weight of each	Planned	Tested	Planned	Tested	Planned	Tested	Planned	Tested	Planned	I from measured section	I from weight-deflection data	Measured strains × 30,000,000
			Lb.	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*In.*	*In.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*
Weighted truss (machine screws)	36′–10″	None	....	1,012.9	1,045.8	0	0	1,012.9	1,045.8	0	0	13,117	12,829	12,581	11,868
		26	19.5	1,012.9	1,045 8	507.0	507 4	1,519.9	1,553.2	2.06	1.836	19,682	19.053	18.685	17,505
		26	34.5	1.012.9	1,045.8	897.0	897.8	1,909.9	1,943.6	3.65	3.266	24,733	23.842	23,381	22,623
Riveted truss (for comparison)	36′–10″	None	....	1,012.9	1,040.9	0	0	1,012.9	1,040.9	0	0	13.117	12,829	12,642	12.750
		26	19.5	1,012 9	1.040.9	507.0	507.4	1,519.9	1,548 3	2.06	1.833	19,682	19,053	18,776	18,930
		26	34.5	1,012.9	1,040.9	897.0	897.8	1,909.9	1,938.7	3.65	3.267	24,733	23,842	23,495	23,670
H-section	36′–10″	None	....	1,012.9	890.0	0	0	1,012.9	890.0	0	0	8,954	8,982	9,428	9,360
		26	19.5	1,012.9	890.0	507.0	507.4	1.519.9	1,397.4	2.06	2.308	13,436	14,103	14.804	14,664
		26	34.5	1,012.9	890.0	897.0	897.8	1,909.9	1,787.8	3.65	4.082	16,884	18,042	18,939	18,729
Weighted truss (machine screws)	28′–4″	None	....	1,379.8	1.403.8	0	0	1.379.8	1,403.8	0	0	13.745	13,246	13,311	13,489
		20	19.5	1 379.8	1,403.8	390.0	390.4	1,769.8	1,794.2	0.72	0.656	17,631	16,930	17,013	17,247
		20	34.5	1,379.8	1,403.8	690.0	690.6	2,069.8	2,094.4	1.28	1.149	20,619	19,762	19,859	20,097
		20	51.75	1,379.8	1,403.8	1,035.0	1,035.4	2,414.8	2,439.2	1.92	1.738	24,056	23,016	23,129	23,462
Rectangular bar	28′–4″	None	....	1,379.8	1,377.6	0	0	1.379.8	1,377.6	0	0	6,140	6,176	6.207	6,345
		20	19.5	1,379.8	1.377.6	390.0	390.4	1,769.8	1,768 0	0.72	0.731	7,876	7,926	7,966	8,124
		20	34.5	1,379.8	1.377.6	690.0	690.6	2,069.8	2,068.2	1.28	1.277	9,211	9,272	9,318	9,480
		20	51.75	1,379.8	1.377.6	1,035.0	1,035.4	2,414.8	2,413.0	1.92	1.905	10,746	10.817	10,871	11,073

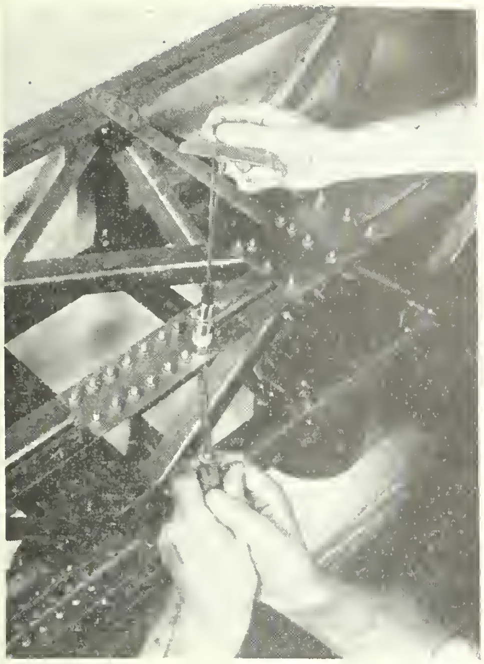


Figure 10.—Use of torque indicating wrench for setting bolt tension.

a straight line if dry friction is the dominant cause.

If the equation for the amplitude decay curve can be determined, the logarithmic decrement value at any oscillation or time value may be obtained directly from the equation. This method was used for determining logarithmic decrement values in all of the tests with the solid sections. An example of the method of analysis is illustrated in figure 11 where amplitude values resulting from experimental measurement are shown as plotted points along an exponential curve, the equation for which was derived from the experimental data. Attention is called particularly to the way in which the observed points lie along the line of the curve, representing the equation derived to fit the observed values. Below in the same figure is another curve showing the variation in logarithmic decrement values as the oscillations damp out, decreasing from an initial value of about 0.0071 to a final essentially constant value of about 0.0031. The decrement values that established this curve were computed from the equation.

In those cases where the damping results from some complex combination of influences it frequently becomes impractical to find an equation that fits the experimentally determined amplitude decay curve. This was found to be the case in a number of tests with the bolted truss. For these data logarithmic decrement values were calculated from the ratio of two successive amplitudes at each of a series of points along the amplitude decay curve.

Logarithmic Decrement Relations

The measurement of both strain and deflection at midspan made it possible to relate extreme fiber stress to midspan deflection for each specimen and from this to relate the instantaneous logarithmic decrement values to maximum fiber stress developed during the particular oscillation to which the values applied. Graphs showing the variation of logarithmic decrement with dynamic stress were particularly useful for the comparisons desired in this investigation and much of the data will be presented in that form.

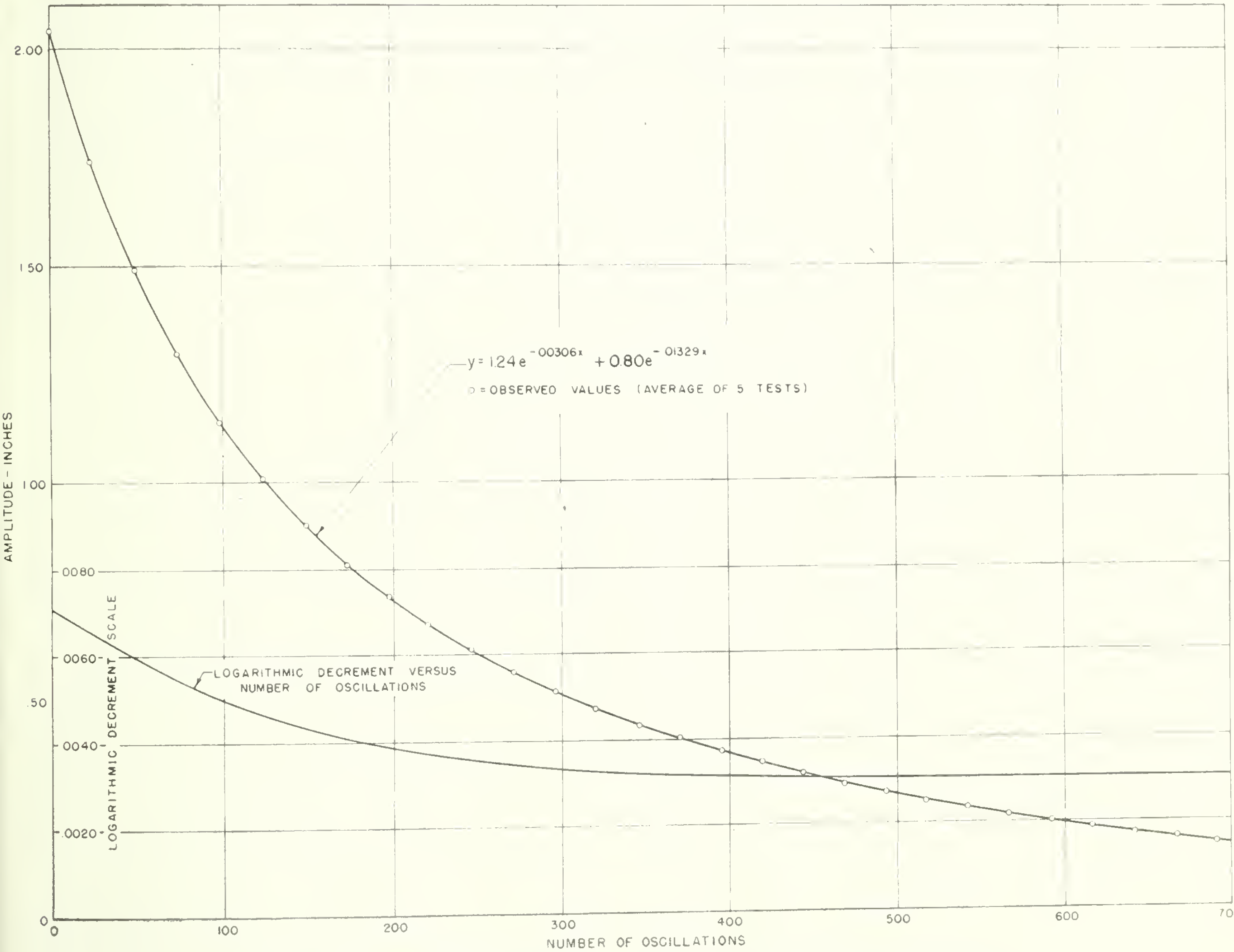


Figure 11.—Analysis of amplitude decay curve: H-section with 34.5-pound weights.

Figure 12 shows the relation between logarithmic decrement and stress for the truss and the **H**-section under identical conditions of test on the 36-foot, 10-inch span as found in the initial tests. In these particular tests the truss was bolted up with the No. 10–24 machine screws drawn up with a 35 inch-pound torque. The initial amplitudes cause the highest stresses and appear toward the right-hand side of the graph. When excited with the 34.5-pound live-load weights the truss showed an initial decrement value of 0.0511 as compared with a value of 0.0071 for the **H**-section. As the amplitudes decreased the dynamic strains decreased and the decrement values decreased also to values of the general order of 0.0048 for the truss and 0.0031 for the solid section of the **H**-beam.

Data from a similar comparison between the shortened truss and the rectangular bar when tested under identical conditions on the 28-foot, 4-inch span are shown in figure 13. Three different live-load weight values were used to excite the specimens in these tests. Even with the 51.75-pound weights the maximum dynamic stress in the bar was only about 4,700 p.s.i. At this stress level the decrement value for the truss was approximately twice that of the solid section of the rectangular bar. In this comparison, as in that between the truss and **H**-section, the maximum damping of the truss was found at the beginning of the oscillations where the amplitude of motion and the stresses were greatest. It is under these conditions that the likelihood of motion in the joints of the truss with the accompanying development of dry or Coulomb friction is greatest. It is to be noted that at low dynamic stress values the logarithmic decrement values are relatively low in all cases.

Each point on these and subsequent figures of the same type represents an average of 5 to 7 tests and may be considered quite well established since the variation for a given set of test conditions usually was quite small.

In these initial tests, with the joints in the truss fastened with the No. 10-24 machine screws, a study was made of the damping of the truss as influenced by the degree of tension in the bolts with which the joints were drawn up. In this study the truss was tested on both the 36-foot, 10-inch span and the 28-foot, 4-inch span and in both cases the oscillations were caused by the release of the 34.5-pound live-load weights. Three degrees of bolt tension, determined by tightening torque values of 15, 25, and 35 inch-pounds, respectively, were used and each of the 1,700 bolts in the truss was adjusted to the value required by the particular test. The relation between logarithmic decrement and dynamic stress for each of the several test conditions is shown in figure 14. From these data it is apparent that the more tightly the joints are

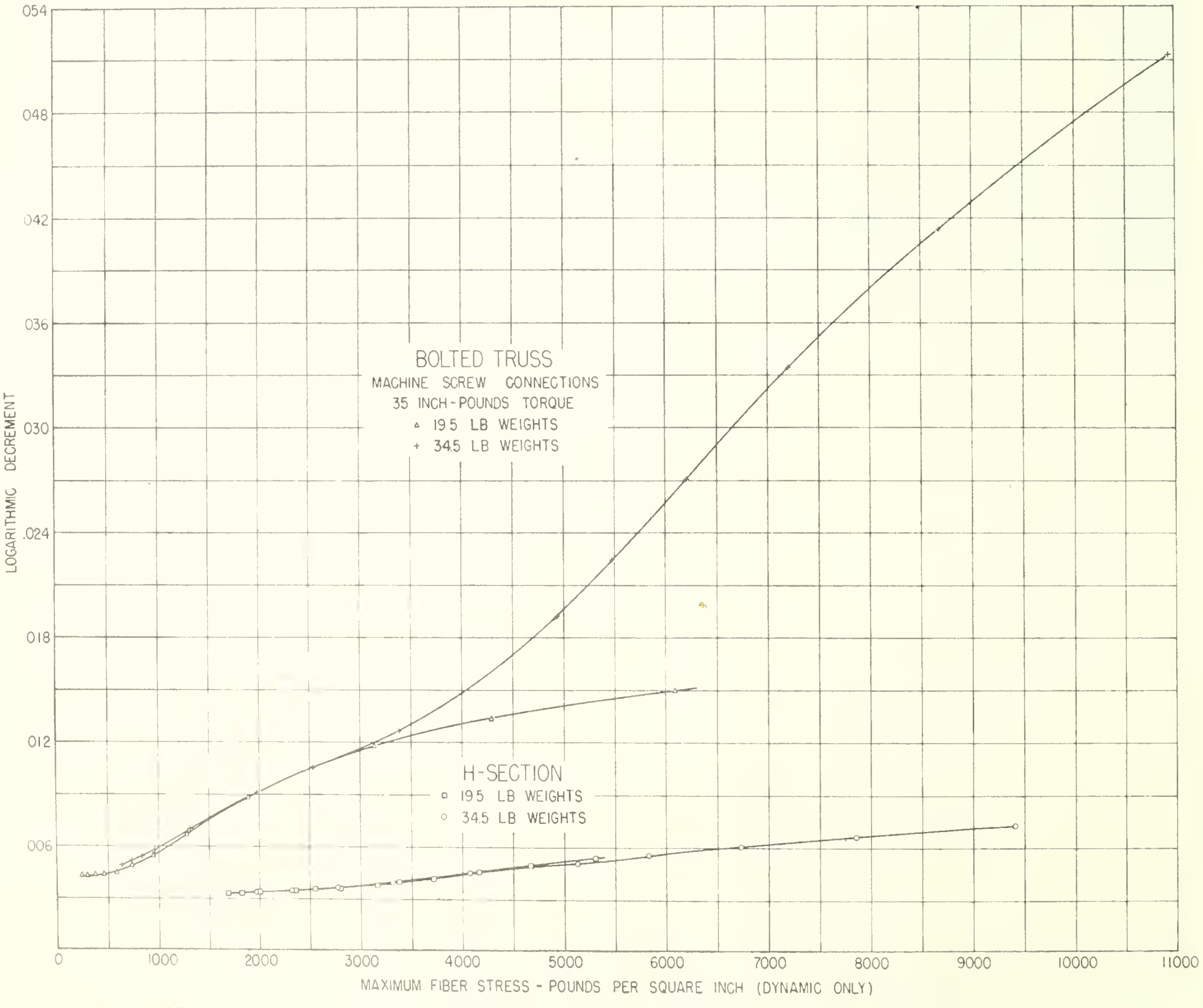


Figure 12.—Variation of logarithmic decrement with stress: Bolted truss and H-section, 36-foot, 10-inch span.

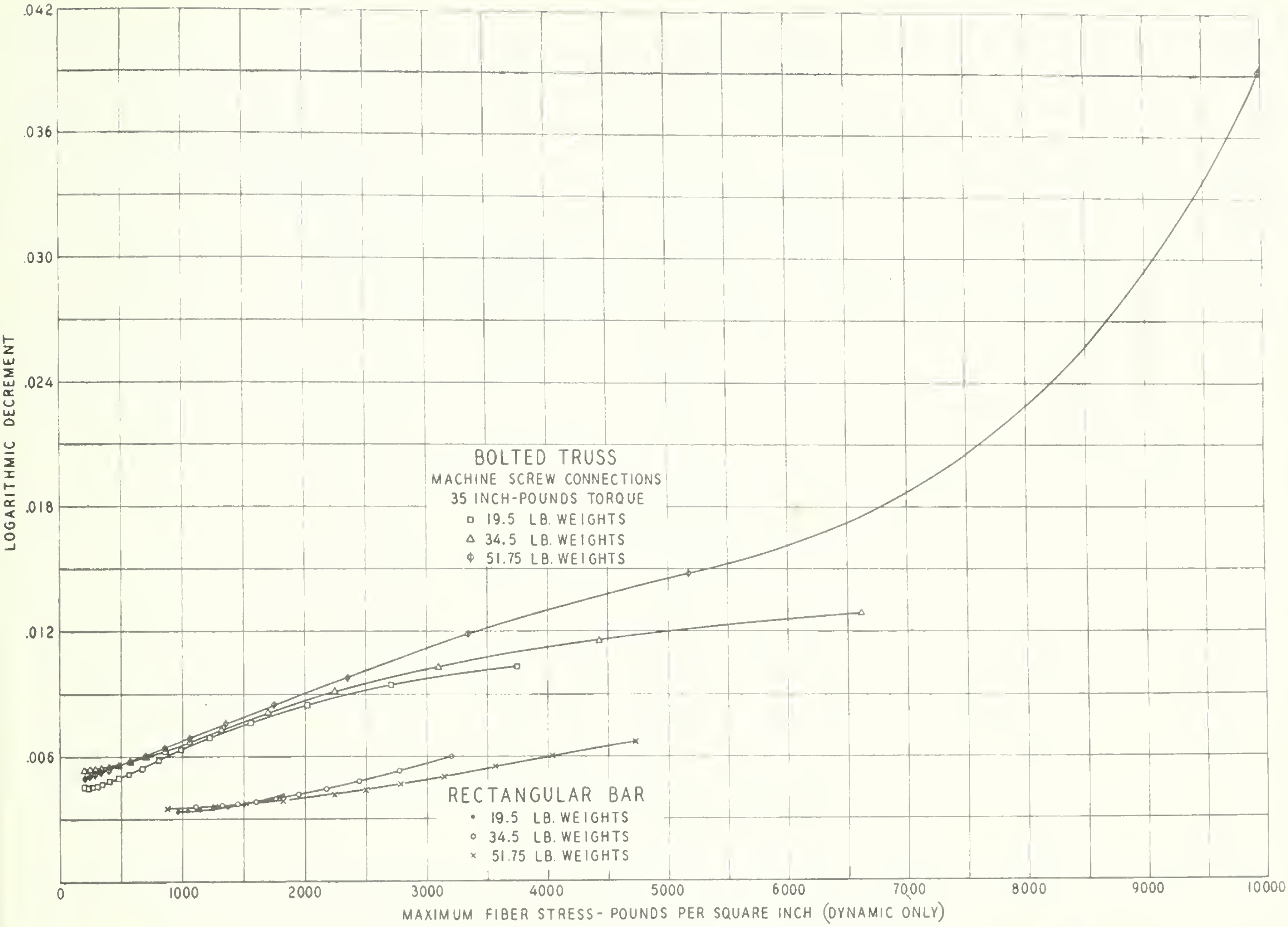


Figure 13.—Variation of logarithmic decrement with stress: Bolted truss and rectangular bar. 28-foot. 4-inch span.

drawn up, the less the damping, particularly at the higher stresses.

Examination of some of the joints near the middle of the truss showed plainly that there had been relative motion between the bolted members and that in some cases the threads along the body of the bolts had been deformed considerably by it. This raised the question as to what might be the damping characteristics of the trussed member if the bolts more nearly filled the holes.

Truss Fastenings Studied

The Advisory Board authorized further tests with bolts that had a solid or unthreaded body where the bolt passed through the angles and gusset plates and that fitted closely in the hole through these members. After some investigation it was found that certain screws available in the aircraft industry had suitable characteristics. These were of high strength steel, manufactured to close tolerances, with well formed threads in the fine series (32 threads per inch). Also, they were obtainable with solid or unthreaded bodies of various lengths. They are known as NAS (National Aircraft Standard) screws and will be referred to as such in this report. The average diameter of those obtained for these tests was found to be 0.186 inch. A special reamer was made with a diameter of 0.187 inch and all holes in splices and other connections of the truss were reamed to this size.

The nuts selected for use with these high strength NAS screws were a commercial product known as the "Elastic Stop Nut." Each nut contains an integral fiber washer which tends to prevent the nut from working loose. Tests with a torque-indicating wrench showed that, as used in these tests, the torque required to turn these nuts on the NAS screws was of the order of 1 to 2 inch-pounds.

Tightening of the NAS screw and Elastic Stop Nut combination showed that failure occurred by yielding of the bolt body in tension at a tightening torque of about 90 inch-pounds, as compared with about 45 inch-pounds for the commercial machine screws used in the original assembly of the truss. In other words, these fasteners were about twice as strong as the screw–nut combination used in the initial tests with the truss.

For the tests of the truss with the NAS screw fastenings, tightening torque values of 15, 25, and 45 inch-pounds were selected. The first two gave direct comparisons with data obtained when the No. 10–24 machine screws were used, while the 45-inch-pound value was chosen because tests showed that it corresponded to a direct tensile stress of about 60,000 p.s.i. in the bolt body. This seemed to be a reasonable upper limit for working stress in structural bolting.

A chord splice made up with the NAS screws tightened with a torque of 15 inch-pounds was tested in tension. As shown in table 3, it failed at a load of 15,220 pounds by rupture of the metal in the angles, the screws being undamaged. In a comparative test with the No. 10–24 machine screws the splice failed at 12,240 pounds by shear in the bolts. The computed stress in the chord angles, based on net section, was about 76,600 p.s.i. in the test with the NAS screws and about 61,500 p.s.i. in that with the commercial machine screws. The appearance of the failed chord section tested with the NAS screws is shown in figure 15.

After all the holes had been reamed and the NAS screws had been installed, tests were made with the truss on the 36-foot, 10-inch span with the 34.5-pound live-load weights, using tightening torques of 15, 25, and 45 inch-pounds, respectively.

The data obtained in these tests are shown in figure 16 and are comparable with those obtained with the truss when fastened with the machine screws as shown in figure 14. It will be observed that with the highest bolt tension, represented by a tightening torque of 35 inch-pounds for the machine screws and 45 inch-pounds for the NAS screws, the relations between logarithmic decrement and dynamic stress are essentially the same.

From this comparison it would appear that insofar as the self-damping characteristics of the truss are concerned the tightness with which the connections are bolted up is of more importance than is the fit of the bolts in the bolt holes.

Tightening procedure examined

At the low tension, represented by a tightening torque of 15 inch-pounds, the truss with the NAS screws showed more damping at the higher stress values than had been found in the comparable tests with the machine screws. This was rather surprising and, in searching for an explanation, it was suspected that variations of this sort, particularly where lower bolt tensions were involved, might be the result of variations in the tightening procedure itself—such as, for example, whether or not a given level of bolt tension had been preceded by a higher or a lower bolt tension value. It was recognized that the relation between bolt tension and tightening torque is susceptible to variation, and throughout the tests great care was used to follow a uniform procedure to control this variation as much as possible.

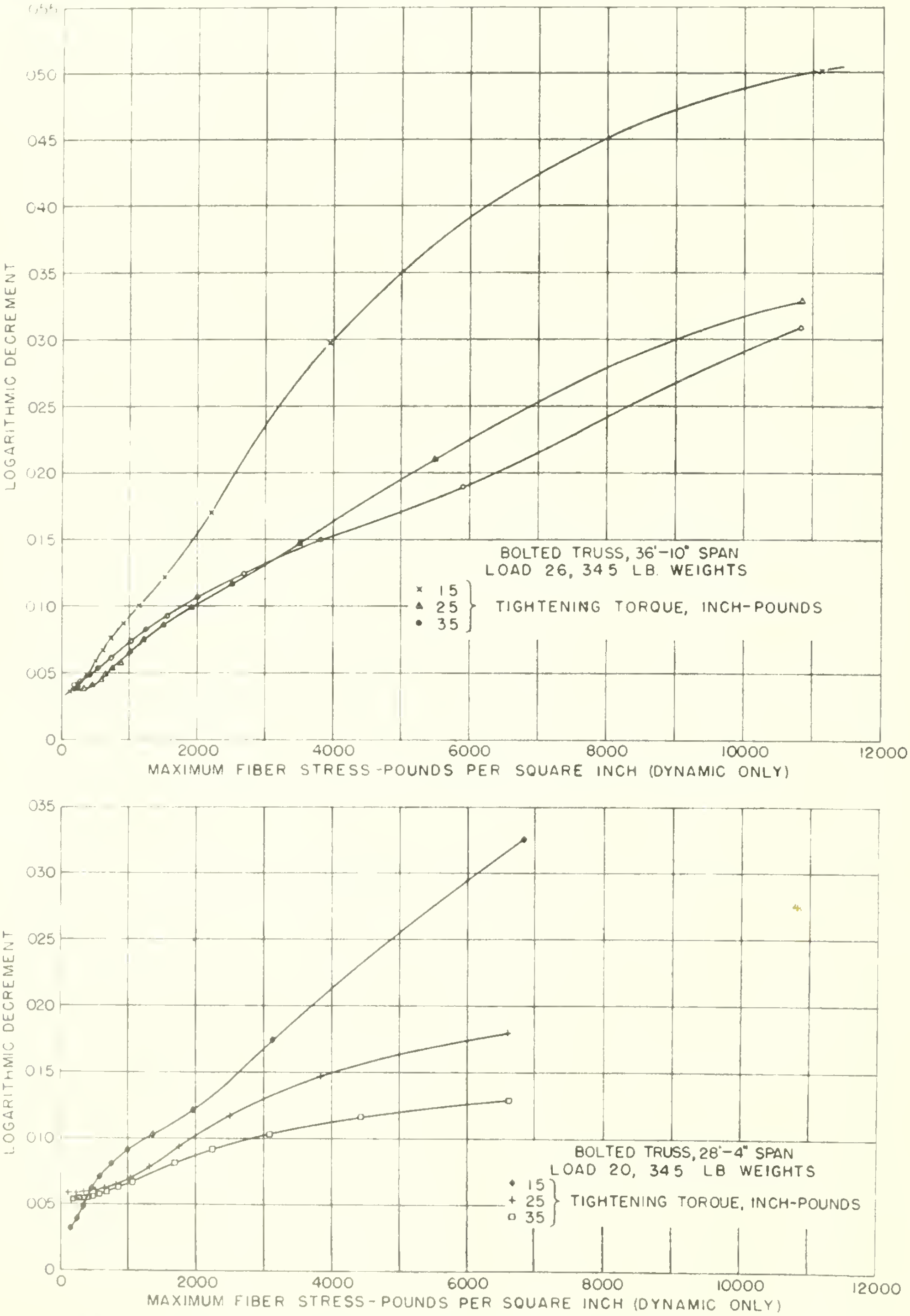


Figure 14.—Variation of logarithmic decrement with fiber stress: Truss (both span lengths) bolted with machine screws with three degrees of bolt tension.

To obtain information on this point, tests were made with the truss on the 36-foot, 10-inch span excited with the 34.5-pound live-load weights. In one series, three bolt tensions were applied in ascending order; in a second series, the same tension values were applied in descending order. The data obtained are shown in figure 17.

It is apparent from these data that the sequence of bolt tightening operations could have a measurable effect on the damping characteristics of the truss, the effect being least at the higher bolt tensions. It is indicated that the more tightly the joints are drawn up, the less sensitive they are to the variations inherent in the tightening procedure. This seems quite reasonable.

It will be recalled, however, that when the chord splices were tested to failure in tension the bolts were brought into bearing and at these high stresses the effect of bolt type and fit in the holes on the ultimate strength of the splice was quite marked.

Effect of Variation in Dead Load

One of the questions raised in the discussion of the damping tests by the Advisory Board had to do with possible effects of changes in dead load on the damping characteristics of the truss. It was particularly desired to learn what might be the effect of an increase in dead load. Because of the relatively high combined stresses developed in the chord members by the dynamic tests, there was a practical limit to the amount by which the dead load could be increased. In order to obtain a relatively wide spread in dead-load values without developing an excessively high combined dead- and live-load stress in the chord members, the dead load of the truss was first reduced from its normal value of 1,059.6 pounds (with NAS screws and Elastic Stop Nuts) to 843.5 pounds by decreasing the dead-load weights attached to the lower chord panel points. After tests had been made with this reduced dead load, using both the 19.5- and the 34.5-pound live-load weights, dead load was added to the truss until it weighed 1,278.2 pounds. The tests were then repeated. Amplitude decay curves for each of the four test conditions are shown in figure 18. The data indicate that increasing the dead load of the truss by 51.5 percent caused no significant change in its damping characteristics.

In the original discussion of the damping test program an item was included that re-

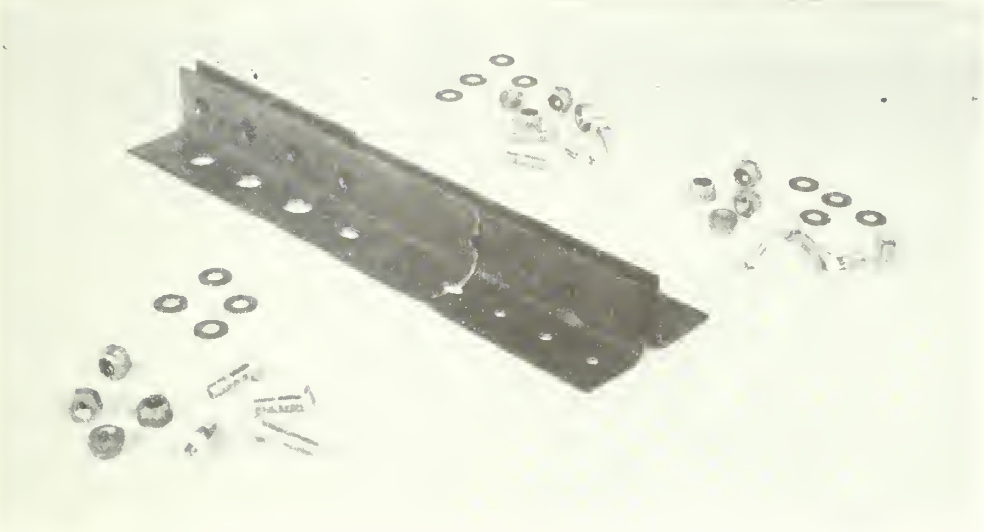


Figure 15.—NAS screws and Elastic Stop Nuts used in truss connections, and chord test section which failed in tension when these screws were used.

lated to the manner of excitation of the specimens. Specifically, the **H**-section was to be excited by the release of live-load weights arranged in two different patterns and varied in amount so as to produce equal deflections of the beam at midspan. One pattern was that used in the tests that have been described—that is, 26 weights, each of 34.5 pounds, uniformly distributed at 17-inch intervals throughout the span length. The other pattern consisted of a series of weights arranged symmetrically about the midpoint of the span length, but not spread uniformly from one reaction to the other.

Use was made of the 51.75-pound weights available from the tests with the rectangular bar. By trial it was found that 12 of these weights would develop the required deflection. The details of the two load arrangements, together with the relations between logarithmic decrement and number of oscillations (or time) as found in each test, are shown in figure 19. From this comparison it may be concluded that insofar as these particular load arrangements are concerned no significant effect on damping characteristics was found.

To summarize: the data obtained in the comparisons of the internal damping characteristics of trussed and solid steel members show that there is a marked difference between the two at the higher vibration amplitudes and higher stress ranges, the damping of the truss being greater than that of the solid section in all tests. At lower vibration amplitudes the differences are less marked and the damping, as measured by the logarithmic decrement values, is relatively quite small with both types of structural members.

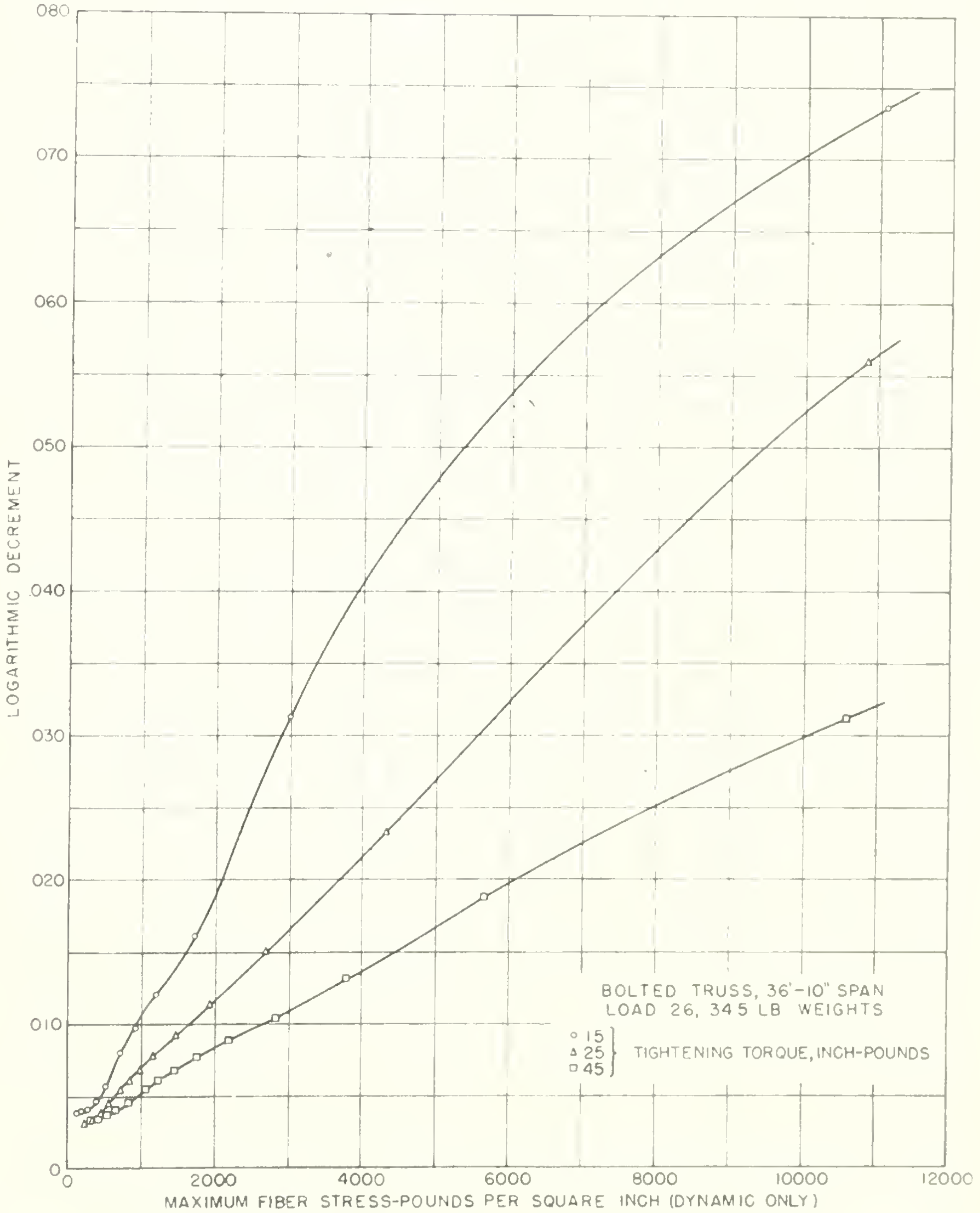


Figure 16.—Variation of logarithmic decrement with fiber stress: Truss bolted with NAS screws with three degrees of bolt tension.

Augmented Damping of Truss

The data obtained in the tests that have been described so far led to a discussion in the Advisory Board of the probable behavior of the truss if it were provided with a simulated deck having sliding bearings to support the deck sections—that is, if the trussed member were made to conform more nearly to the deck structure of a suspension bridge. As a result of the discussion it was decided to extend the program to include some study of the effects of augmenting the dry friction within the truss with dry friction developed by mechanical brakes, by sliding bearings at the ends of sections of a simulated floor system, and by sliding bearings at the truss support. These studies will be discussed in the order named.

Tests with Mechanical Brakes

It was considered desirable as a part of the program in which the damping of the truss was augmented by dry or Coulomb friction, applied externally, to have some system by which the amount of the frictional force could be controlled within close limits.

For this purpose a mechanical brake designed to develop known frictional forces

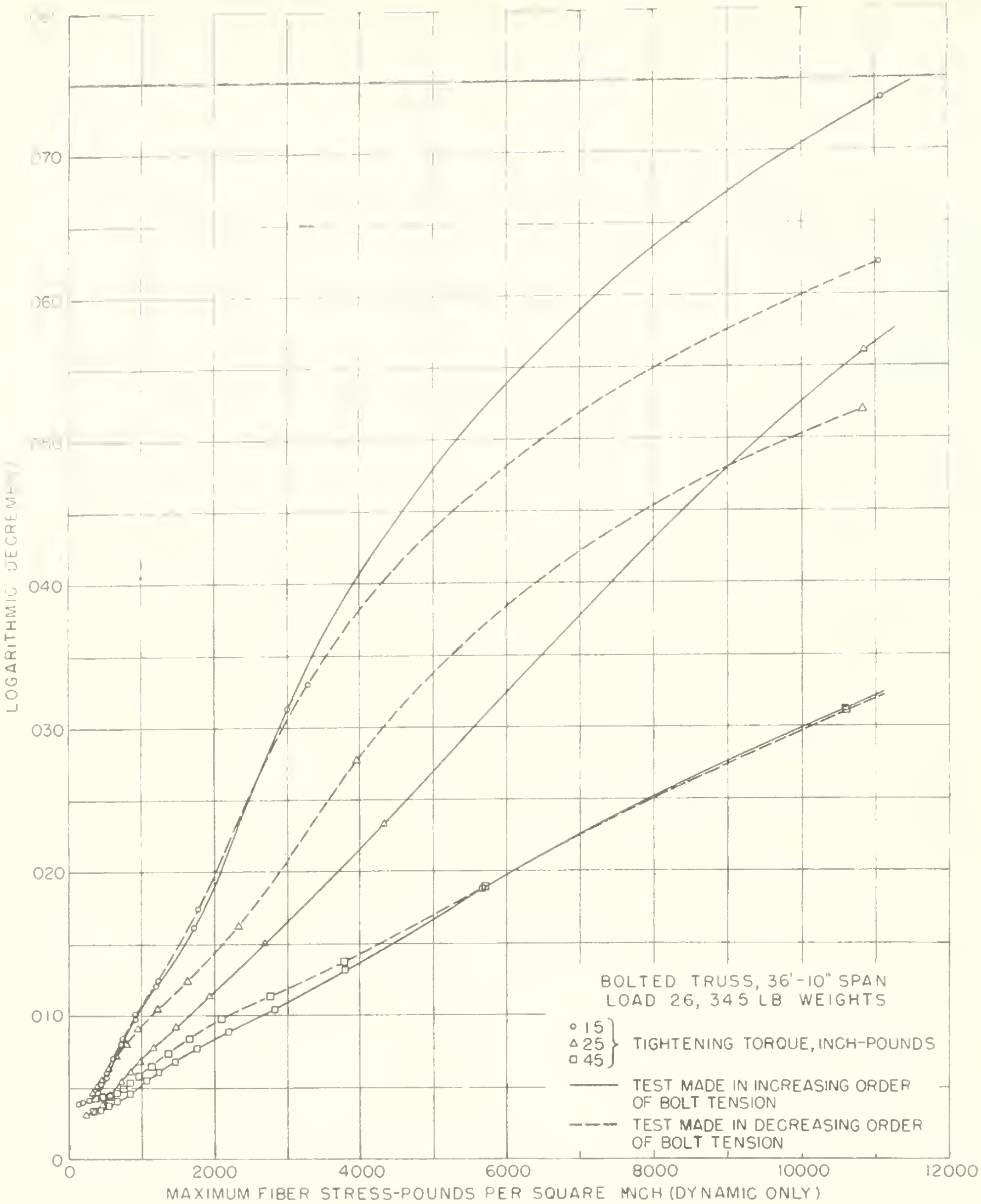


Figure 17.—Effect of bolt tightening sequence on logarithmic decrement–fiber stress relations.

at the ends of the truss was constructed. The principle of the brake is quite simple and its operation may be readily understood by reference to figure 20. To the inner face of the concrete pedestal that carried the end bearings of the truss a steel support or post was bolted. This post extended upward through the truss and provided a ball-bearing fulcrum for a horizontal lever that extended outward longitudinally over the end frame of the truss. At the outer end of the lever there was suspended on knife edges a pendant platform on which cast-iron weights could be applied to develop a known downward force on friction surfaces located beneath the lever and at a point near the fulcrum. These surfaces were of several forms and materials and will be described presently. The upper surface was attached to the lever while the lower surface was carried on a plate bolted transversely across the truss in the plane of the upper chord and directly over the end frame. The downward reaction at the friction surfaces was, therefore, directly over the end bearing of the truss.

As the truss oscillated in the vertical plane the end frame went through a rotational motion, causing a horizontal sliding motion at the plane of contact of the friction surfaces. With the three weights used on the platform of the brake, vertical reactions of 70, 140, and 210 pounds were developed on the braking surfaces. One of these brakes was installed at each end of the truss and both were operative in all of the tests in which brakes were used.

Friction surfaces varied

In the initial tests with the brakes, both of the friction surfaces were of cold rolled steel—the upper surface being a cylindrical segment of 6-inch radius, the lower surface being plane. These were the surfaces in use when the photograph of figure 20 was taken. To insure dry friction the rubbing surfaces were carefully cleaned before each test.

It was at once evident that the rubbing of a cylindrical surface of cold rolled steel on a plane surface of the same material in the absence of a lubricant was causing a surface scoring or "galling" which resulted in erratic test results and was, therefore, not satisfactory for comparative testing. The condition was present in tests at relatively low pressures on the friction surfaces, but was more severe at the higher pressures.

To increase the areas of contact of the friction surfaces and thus decrease the pressure intensity, circular plane surfaces of 1.0- and 2.0-square-inch area, respectively, were made up as replacements for the upper cylindrical surface used in the initial tests. These also were of cold rolled steel, finely finished. They were conical in form and were fitted with a ball seat at the top to insure that the plane friction surfaces would remain in uniform bearing as motion occurred. Even with the greatly reduced pressure intensities obtained by this modification the scoring continued as long as the opposed surfaces were of cold rolled steel and were without lubrication.

The character of the material used for the lower friction surface was next changed. Inserts of bronze or of hardened steel were fastened in the plate that was bolted across the truss. The bronze insert was made of some bridge bearing material left over from an earlier research program and met the requirements of A.S.T.M. Designation B-22-21, Class C. The steel insert was of flat drill-rod stock hardened to Rockwell C 57.

These various friction surfaces are shown in figure 21. In the bottom row from left to right are the 2-square-inch circular plane surface, the bronze insert, the hardened steel insert, and the 2-square-inch circular plane surface in its normal position with the ball in place to bear against the lower surface of the lever arm. In the upper row of the group are two views of the cylindrical steel surface used in the initial tests with the brake and of the 1-square-inch circular plane surface of cold rolled steel used later.

Damping effect of brakes

With the friction brakes in operation, the oscillations of the truss damped out much more rapidly than did those obtained under identical test conditions without this externally applied frictional force. This is as would be expected. Because of this rapid rate of amplitude decay it was no longer feasible to determine the logarithmic decrement from the intermittent records of movement obtained with the stylus at midspan and recourse was had to a magnetic type pickup with oscillograph recording, as mentioned earlier. This type of recording was used in all of the tests in which external friction was applied to the truss. Typical records obtained with this recording system are shown in figure 22. These records are of interest because they indicate very clearly the strong damping developed by the brakes, particularly as the braking force F_n is increased, and also because they show the difference in shape of the envelope curve where the damping is principally caused by dry or Coulomb friction.

Amplitude decay curves obtained with the mechanical brakes in operation are shown in figures 23 and 24 in comparison with that for the truss when no external friction was present. Figure 23 contains data obtained with the bronze lower friction surface in contact with cold rolled steel upper friction surfaces of each of the three types and for three pressure intensities. Figure 24 contains data for comparable test conditions except that the lower friction surface was of hardened steel.

In these figures and some that appear later the amplitude values are as measured with the pickup unit near the end of the truss. They are, therefore, only about 10 percent of the corresponding amplitude values at midspan.

It is apparent that where the bronze and steel plates were used as opposing friction surfaces (fig. 23) the braking action was smooth and consistent, the damping increased progressively with the increase in force applied normally to the surface (F_n), and the area of the friction surfaces had little influence. In other words, the action conformed to the basic principles of dry friction.

In contrast, where the opposing friction surfaces were of steel, even though one was much harder than the other, abrasion and scoring were prevalent. The result, as will be seen from figure 24, is that although the damping action was at times greater than with the bronze-steel combination, the action was neither consistent nor dependable. Although the damping does increase with increases in the force applied normally to the friction surfaces, the relation is not particularly consistent. It is apparent also that with the greater pressure intensities obtained with the smaller contact areas the frictional resistance was greater than with the larger areas, indicating that smooth sliding friction did not obtain.

It is of particular interest that the observed behavior of these friction surfaces was quite similar to that observed in a series of friction tests on sliding bearings made by the Bureau of Public Roads a number of

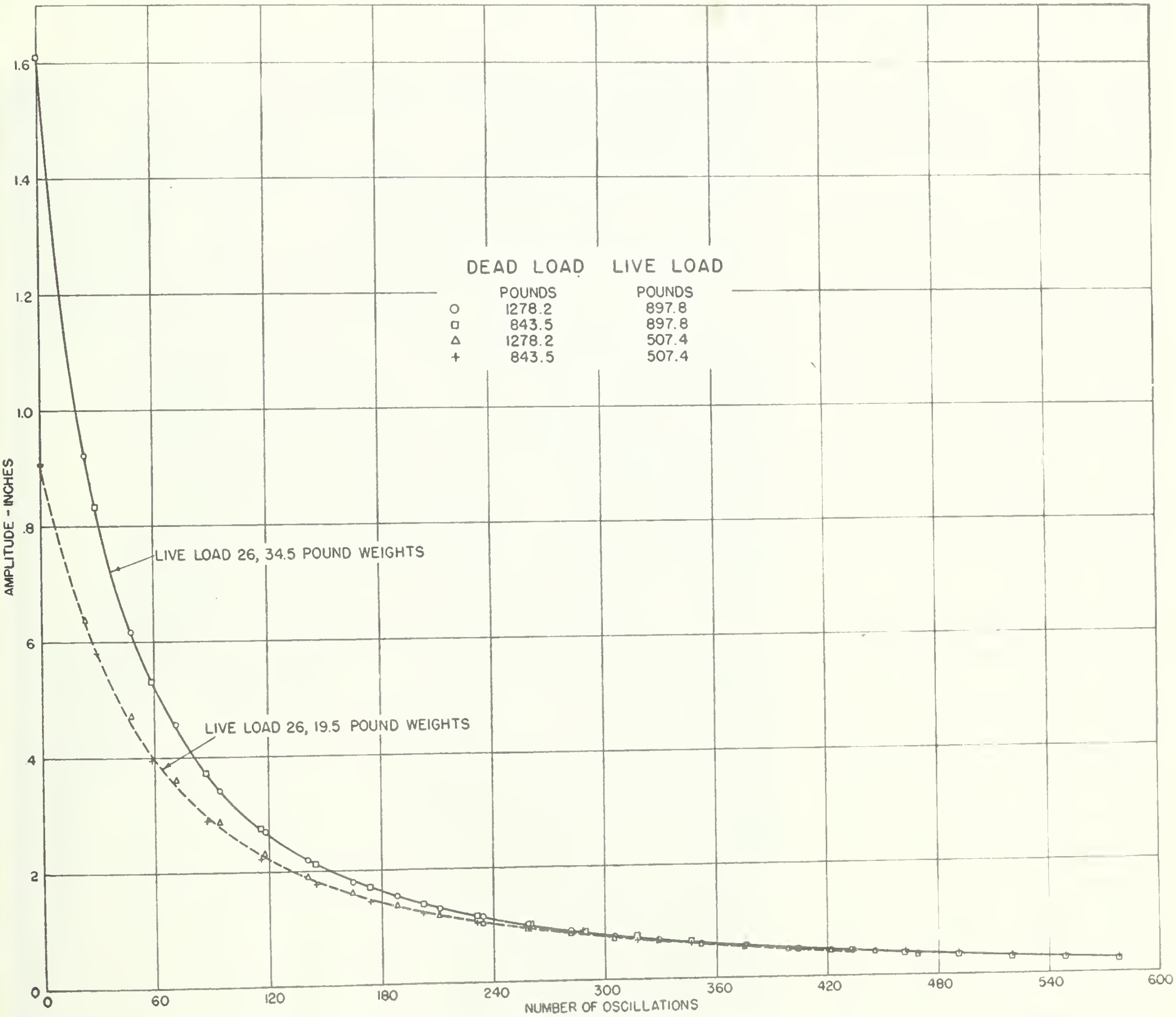
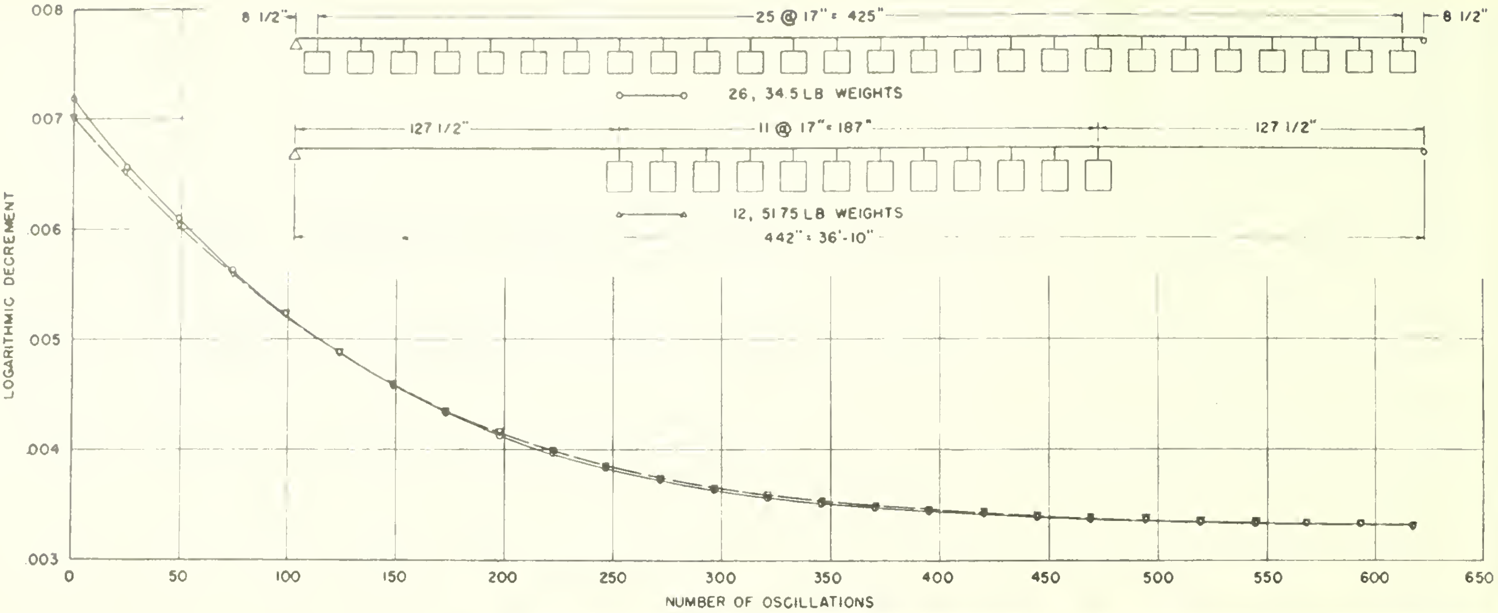
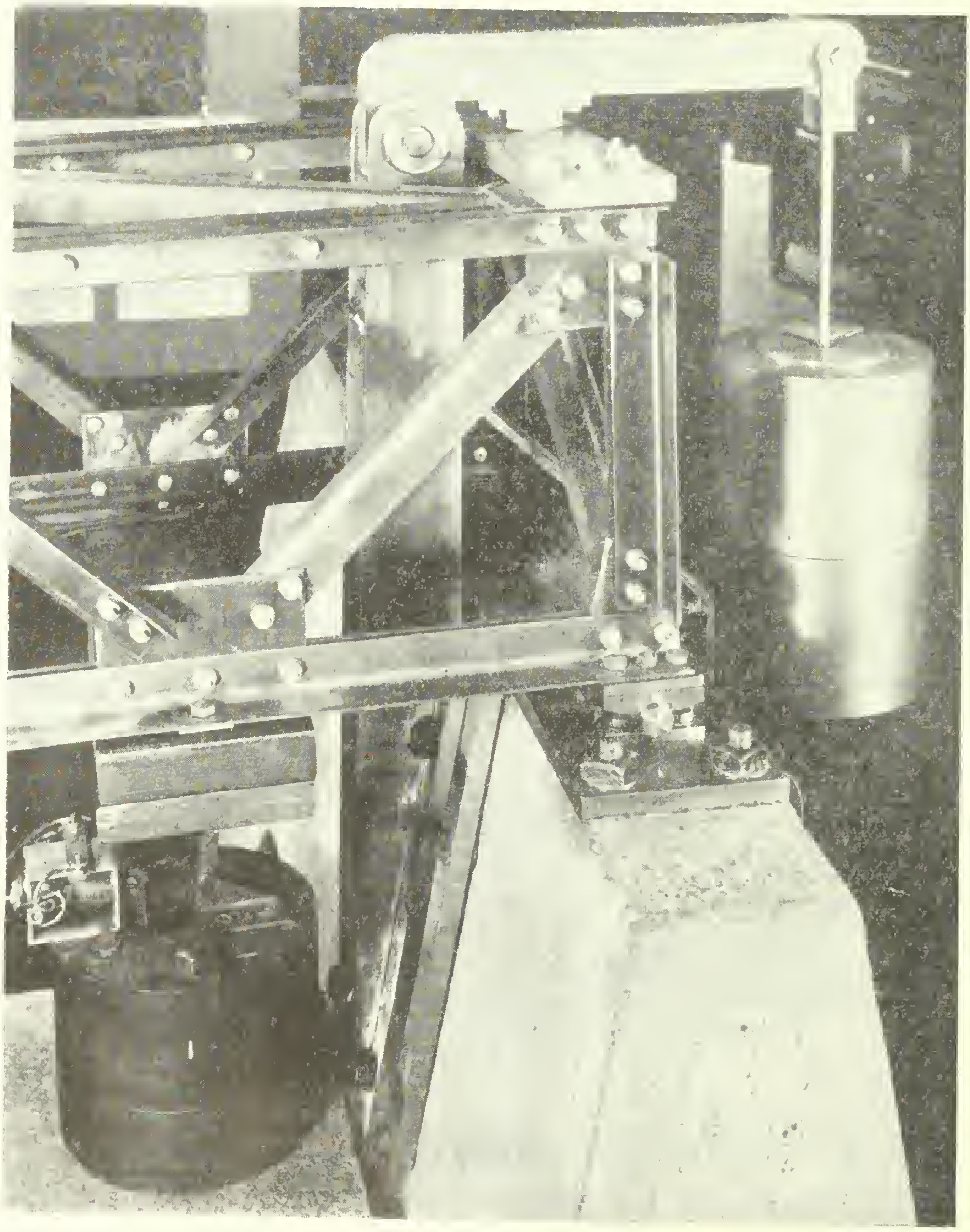


Figure 18.—Effect of dead load variations on amplitude decay: 36-foot. 10-inch span truss bolted with NAS screws with 45 inch-pounds tightening torque.



(ABOVE) *Figure 19.—Comparative effects of uniform loading and of symmetrical but non-uniform loading on the damping of the H-section (midspan deflection the same in both cases).*

(RIGHT) *Figure 20.—Friction brake, with cylindrical surface on brake lever in contact with plane surface on truss.*



years ago.[4] It was concluded from these earlier tests that combinations of like or unlike ferrous metals gave the highest coefficients of friction and that ferrous metals in contact with bronzes gave the lowest. Also, where the opposing plates were of ferrous metals, either like or unlike, seizure and scoring almost always occurred.

For the comparative tests required in this program it seemed clear that a combination of steel and bronze must be used for the friction surfaces if the data were to be concordant and susceptible of analysis, even though the frictional forces developed were somewhat smaller.

With the mechanical brakes acting, it is evident from the data in figure 23 that the oscillation amplitudes decrease very rapidly to values of the order of 0.01 inch or less, as measured at the deflection gage position near the end of the truss (which corresponds roughly to amplitudes of 0.1 inch or less at midspan). Also it is apparent that the shape of the envelope or amplitude decay curve tends to become linear as the braking force is increased. This is as would be expected, since the Coulomb damping supplied by the braking system tends to become the dominant component as this force is increased.

In figure 25 are shown logarithmic decrement values computed from the amplitude decay curves of figure 23 for the 2-square-inch friction plate area and for four values

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[4] *Determination of coefficients of friction of sliding bearings for bridges*, by G. W. Davis. PUBLIC ROADS, vol. 17, No. 10, Dec. 1936.

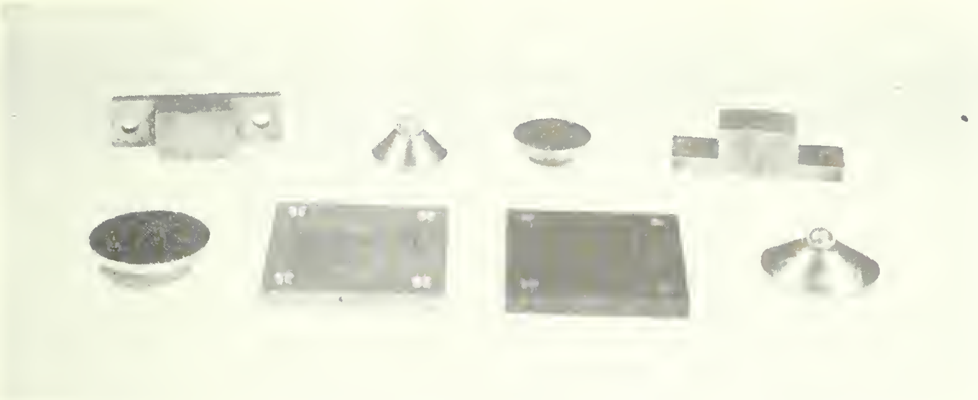

Figure 21.—Various bearing surfaces used in the friction brakes.

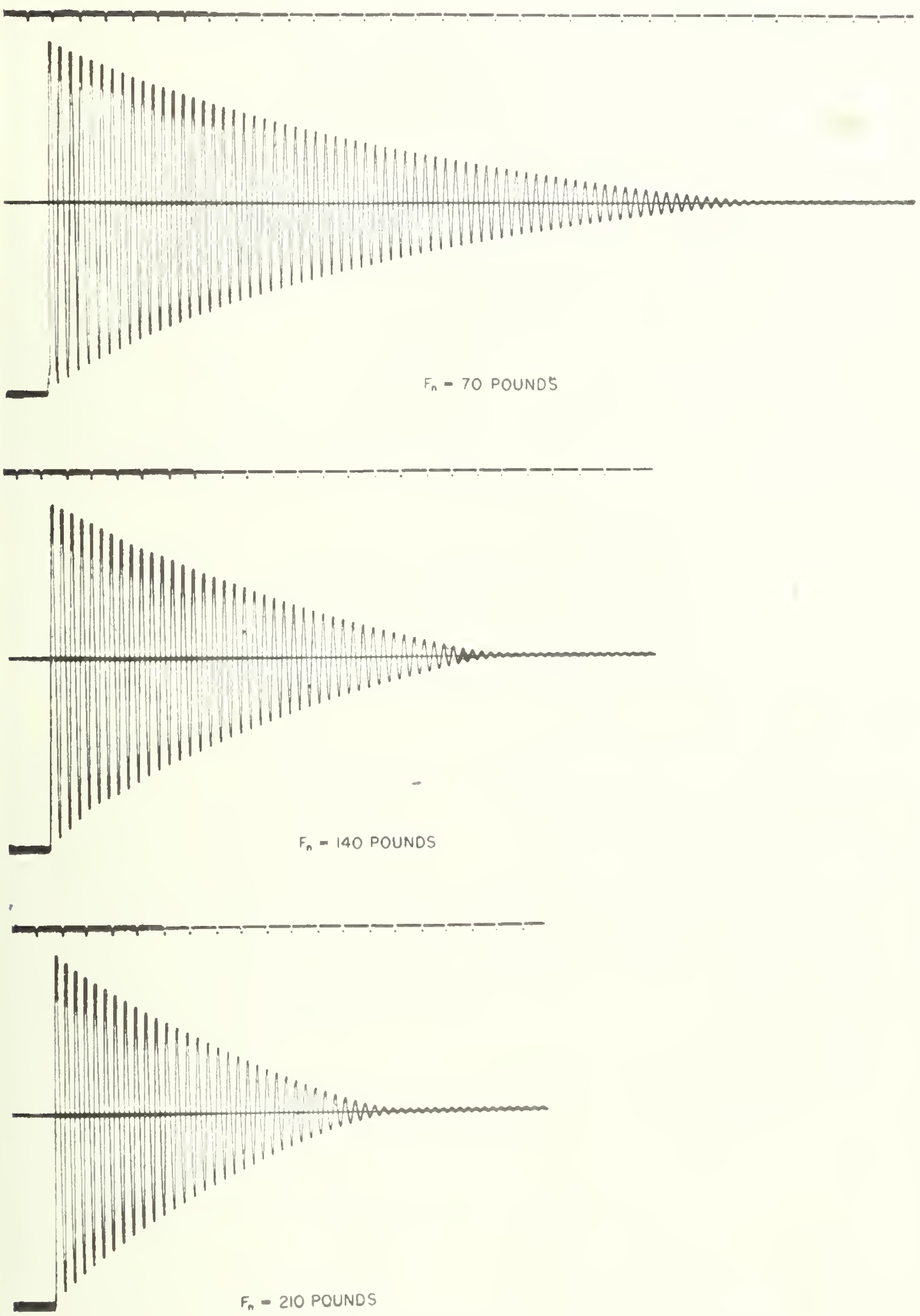

Figure 22.—Typical amplitude decay records of friction brake tests, with 1-square-inch plane contact area.

of the braking force as measured by the normal component F_n. The complex nature of the overall damping with the brakes in operation made it expedient to compute decrement values directly from successive measured amplitudes at frequent intervals, as described earlier. This accounts for the slight scatter of the points for the three upper curves in figure 25.

In the case of Coulomb damping the logarithmic decrement is inversely proportional to amplitude. It can be shown that such a hyperbolic relation exists in the data from the tests in which the maximum braking force was used, that is, $F_n = 210$ pounds, which indicates that with the highest braking force active, the damping is almost entirely from the braking action, hence follows the laws of dry or Coulomb damping.

At lesser values of F_n the action is not so completely dominated by the dry friction developed by the brakes and the damping relations are more complex.

Simulated Floor System

The second group of experiments involving the development of additional Coulomb damping were those in which a simulated floor system, with sliding bearings at intervals, was placed on the top of the trussed member.

This deck was designed as a series of steel plates 7¾ inches wide by 3/16 inch thick, arranged end to end over the full length of the truss. Except for one section 34 inches in length at the center of the span, the deck sections were each 8 feet, 6 inches in length. At alternate panel points of the upper chord small plates of cold rolled steel were bolted transversely on the truss and on these the steel-plate deck sections were supported. At one end of each individual deck section (except the 34-inch section in the center) two short vertical legs were fastened. This pair of legs was attached to one of the transverse plates by a hinge or knuckle connection that was resistant to horizontal or to vertical motion, but not to rotation. At each of the other support points, the deck section was fitted with a similar pair of legs except that these were tipped with bronze shoes at the lower end to create sliding friction bearings for the horizontal movements that occurred at these points as the truss oscillated in the vertical plane.

A deck load of about 500 pounds was considered desirable for the purposes of the tests. To develop this and at the same time to maintain the total dead-load weight that had been used in the majority of the tests with the truss, the dead-load weight attached to the truss at the lower panel points was reduced from 768.9 pounds to 274.3 pounds, a difference of 494.6 pounds. Ballast weights attached to the deck sections were used to increase the weight of the simulated floor system by an amount which equaled approximately that removed from the lower panel points. The steel

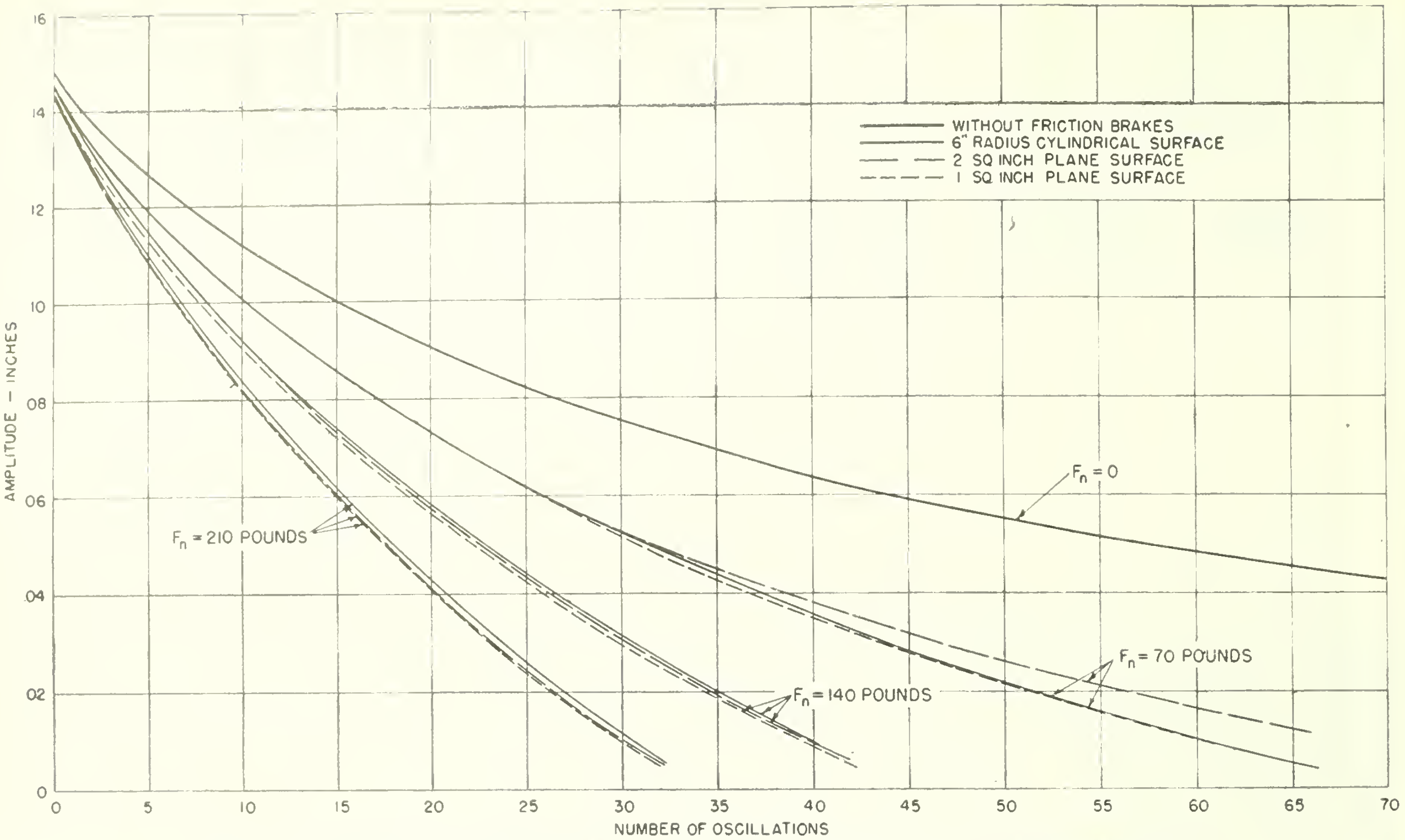

Figure 23.—Effect of various degrees of dry friction on amplitude decay: steel against bronze.

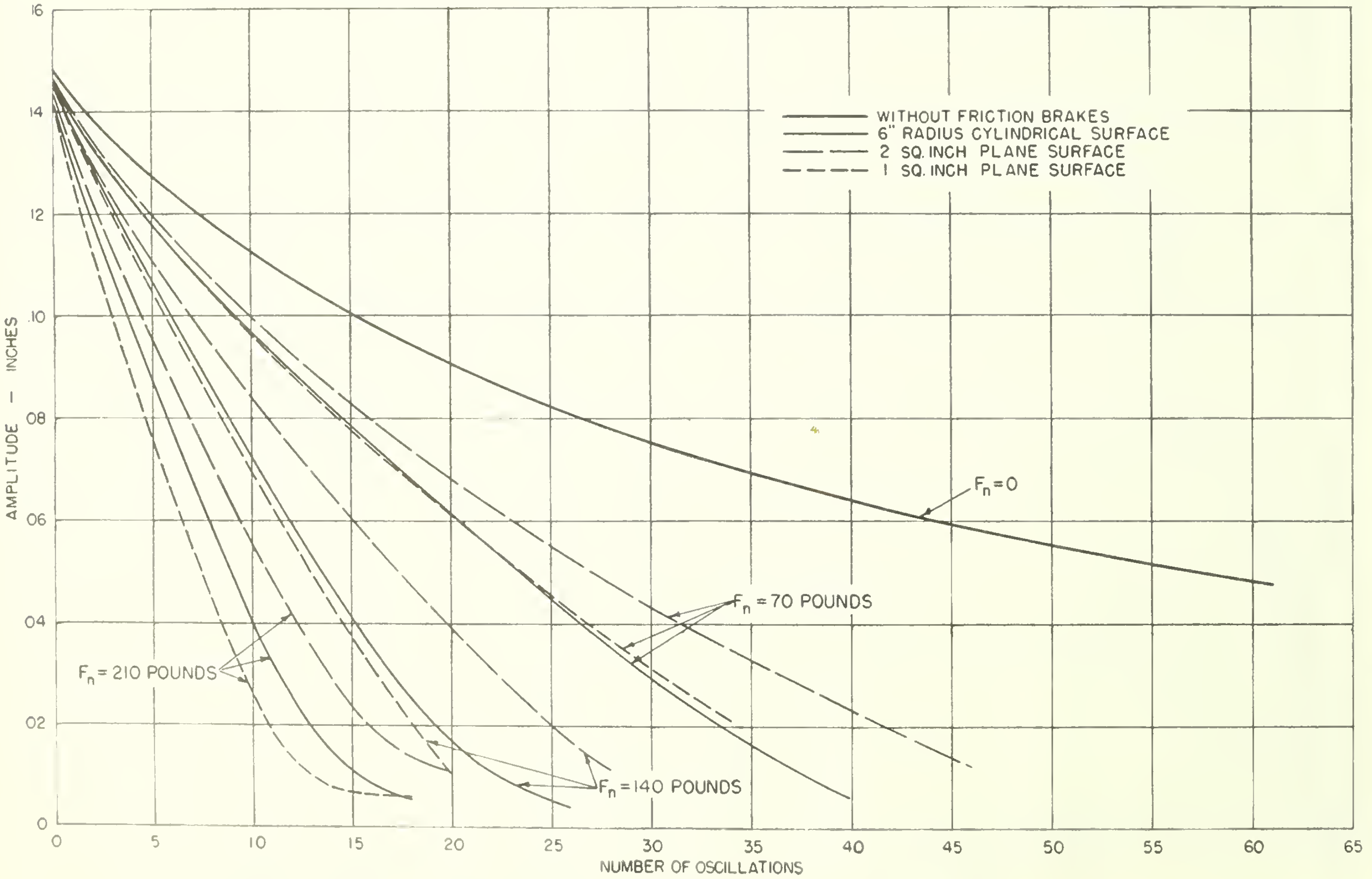

Figure 24.—Effect of various degrees of dry friction on amplitude decay: steel against steel.

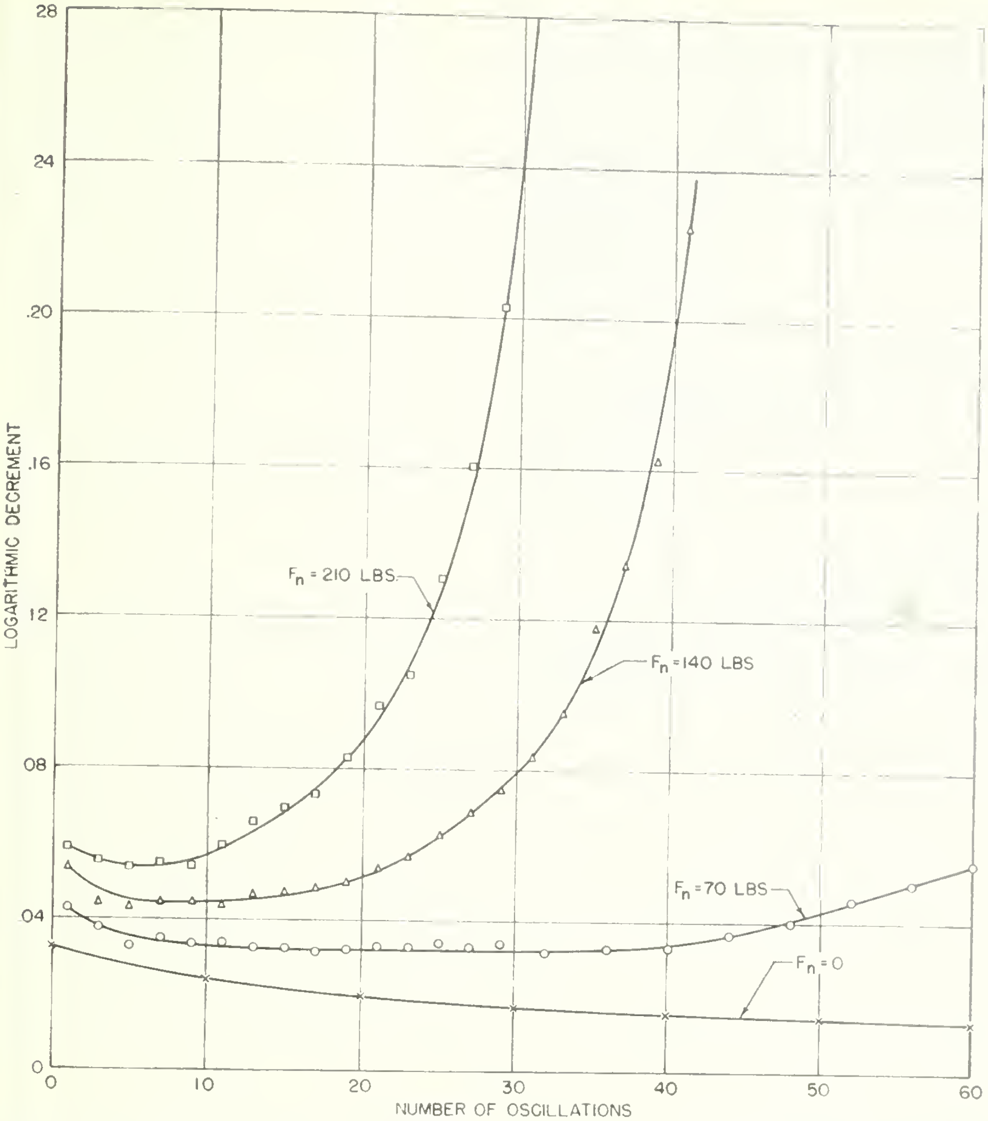

Figure 25.—Effect of various degrees of dry friction on logarithmic decrement values (36-foot, 10-inch truss loaded with 26 weights, each of 34.5 pounds, and bolted with NAS screws at 45 inch-pounds tightening torque; brakes steel against bronze. 2-square-inch plane areas).

deck, supports, and ballast weighed 497.7 pounds, making the total weight of the specimen 1,062.6 pounds as compared with 1,059.6 pounds before the deck was added.

Figure 1 (p. 203) shows the general appearance of the truss with the deck sections in place. In the foreground may be seen the electromagnetic pickup device used for recording oscillation amplitudes in this part of the program. In figure 26 are shown the details of the end connection of one deck section, the sliding bearing at the end of the adjacent section, and the manner in which the ballast weight was applied to the plates over the support points so that there would be no tendency for a secondary period of vibration to develop in the deck sections as the truss oscillated. The bronze tipped legs supporting the deck sections rested in shallow flat-bottomed grooves machined in the plates that were bolted to the truss. These also are shown in figure 26.

The 34-inch section in the center of the span was equipped with friction shoes at both ends rather than being fastened to the truss at one end as were the other sections.

Joint openings of ⅛ inch between sections were provided to insure freedom for longitudinal movement.

Air damping studied

It was recognized from the beginning that in all of the tests a certain amount of air damping was present. However, there was evidence that it contributed only in a small degree to the overall damping for which the logarithmic decrement was being determined.

With the addition of a relatively large horizontal area such as was represented by the plates of the simulated deck, there was reason to suspect that the air damping might become a significant component of the overall damping and some experiments were made to determine how important this influence might be.

A series of plates of ¼-inch plywood were attached to the truss as a replacement for the steel plates of the deck system. Each plywood plate was attached to the truss at the midlength of the plate and the plates were separated slightly so that no stiffening effect was introduced. The total area of plywood plates was the same as that of the steel deck. The total weight of the plywood was 16.75 pounds. A wooden strip was used at the center of each section to support it and to raise it clear of the steel of the truss.

There was noticeable fanning of the air when the truss oscillated, particularly where the amplitudes were large. By generating chemical smoke underneath the truss, the puffs of air expelled at the edges of the plates were made quite apparent.

Amplitude decay curves for the truss with and without the plywood plates attached are compared in figure 27 with a similar relation obtained from tests of the truss with the steel deck sections in place. It is apparent that the additional air resistance created by the application of the plywood plates and the consequent addition of approximately 24 square feet to the frontal area did increase the overall damping to a degree that could be measured, although the effect was relatively small. This tends to support the assumption that, in the case of the truss without a deck, air resistance was not an important contributing component to the damping.

Damping effect of deck

With the steel deck sections added and Coulomb friction active at the sliding deck support bearings, the increase in damping was relatively large (fig. 27), being quite similar to the effect observed with the brakes acting at the ends of the truss, the magnitude corresponding approximately to that observed with a braking force $F_n = 70$ pounds.

The variation of logarithmic decrement value with dynamic fiber stress for three deck conditions (no deck, plywood plates, and steel deck with deck support friction) is shown in figure 28. As in other graphs of the same type, the high stress values measured at the beginning of the test are at the right-hand side of the graph. The decrement values were computed from measured values of successive amplitudes in the manner previously described, and again the slight scatter of the points on the upper curve is attributed to the difficulty in determining decrement values precisely by this method where the damping rate is high.

In figure 29 are shown logarithmic decrement values plotted against the number of oscillations (or time) for the three deck conditions and for two live-load weight values used in the tests. This figure is similar to figure 25 except for the extent of the horizontal scale.

It is interesting to note that the increase in the value of the logarithmic decrement with decrease in vibration amplitude that was observed in the tests with the friction brake is present also in these data from the tests with the steel deck in place. The upward trend of the decrement values reaches a maximum after about 45 oscillations with the smaller live load and after about 65 oscillations with the larger live

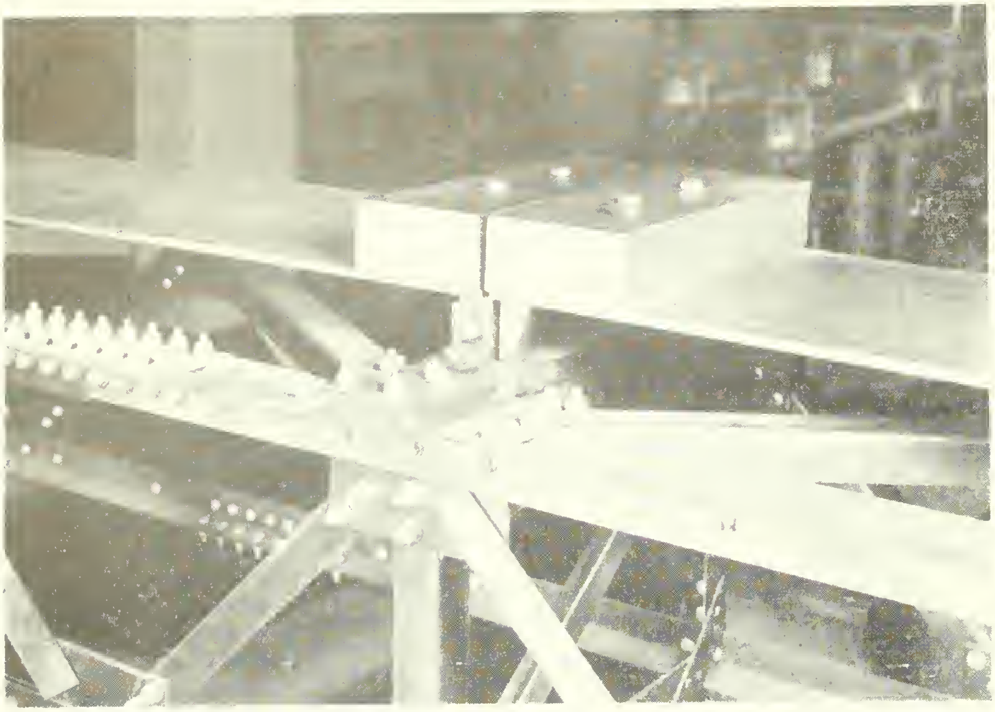

Figure 26.—Fixed and sliding bearings at ends of steel plates of simulated deck placed on top of the truss.

load, after which the decrement values decrease very rapidly. These peak values are significant. It will be noted from figure 27 that after 65 oscillations the amplitude of vibration is quite small (approximately 0.1 inch at midspan). When the amplitude decreases to this value, sliding of the friction bearings of the floor system ceases and from that point on damping is controlled by the conditions of static friction.

As was to be expected, the frequency of oscillation of the truss changed slightly after the steel deck sections were installed. Also, it was found that the change in frequency was not constant but tended to increase slightly as the amplitude became smaller. For example, from seven tests with the larger live load (26 weights of 34.5 pounds each), the average frequency during the first 150 oscillations was found to be 2.5913 vibrations per second while the average for the remaining portion of the records was 2.6275 vibrations per second. Using these values to evaluate the effect of the deck on truss stiffness, it was found that the apparent moment of inertia ranged from 20.99 inches ^{4} (average for the first 150 oscillations) to 21.58 inches ^{4} (average for the remaining oscillations). The value of the moment of inertia of the truss without the deck, as given in table 1, is 20.7 inches ^{4}. From these comparisons it can be concluded that the effect of the steel deck and the forces developed at the sliding shoes of the deck sections was to add slightly but measurably to the stiffness of the truss.

Sliding End Bearings

The third method used in these studies for developing external Coulomb friction was the substitution of sliding bearings for wheel bearings at one end of the specimen. This method was applied both to the bolted truss and to the **H**-section. To make the desired comparisons, sliding shoes were designed that would mount on the same cross shaft that normally carried the ball-bearing wheels. The shoe body was of steel and served as a housing for a self-alining ball bearing that slipped over the cross shaft. The lower edge of this steel body was faced with a bronze shoe plate splined to it. This shoe plate, 3.0 inches long and 0.6 inch wide, served as the moving friction surface. It rested on a finely

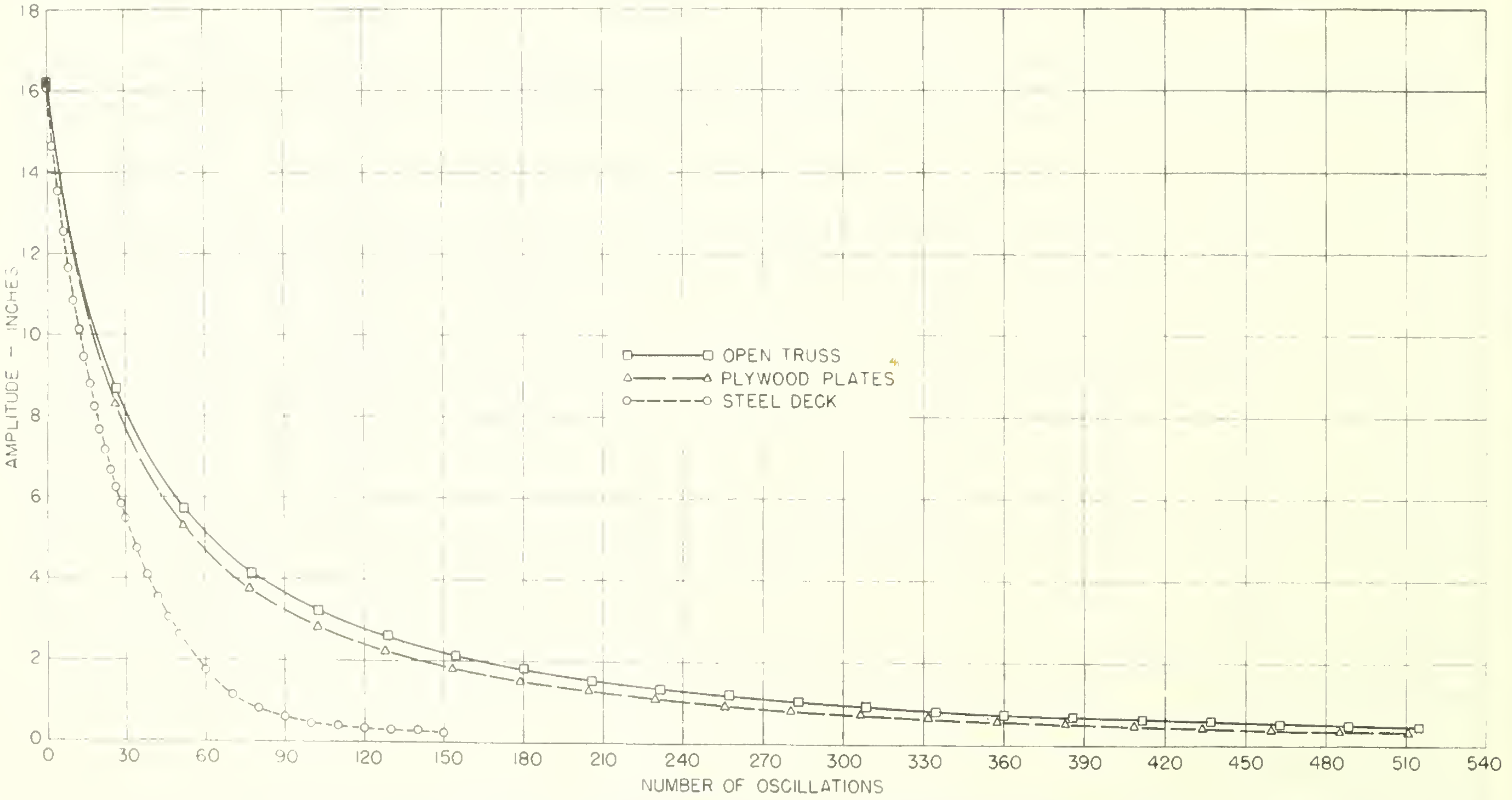

Figure 27.—Effect of variations in deck condition on amplitude decay (36-foot, 10-inch truss loaded with 26 weights, each of 34.5 pounds, and bolted with 4-48 screws at 15 inch-pounds tightening torque).

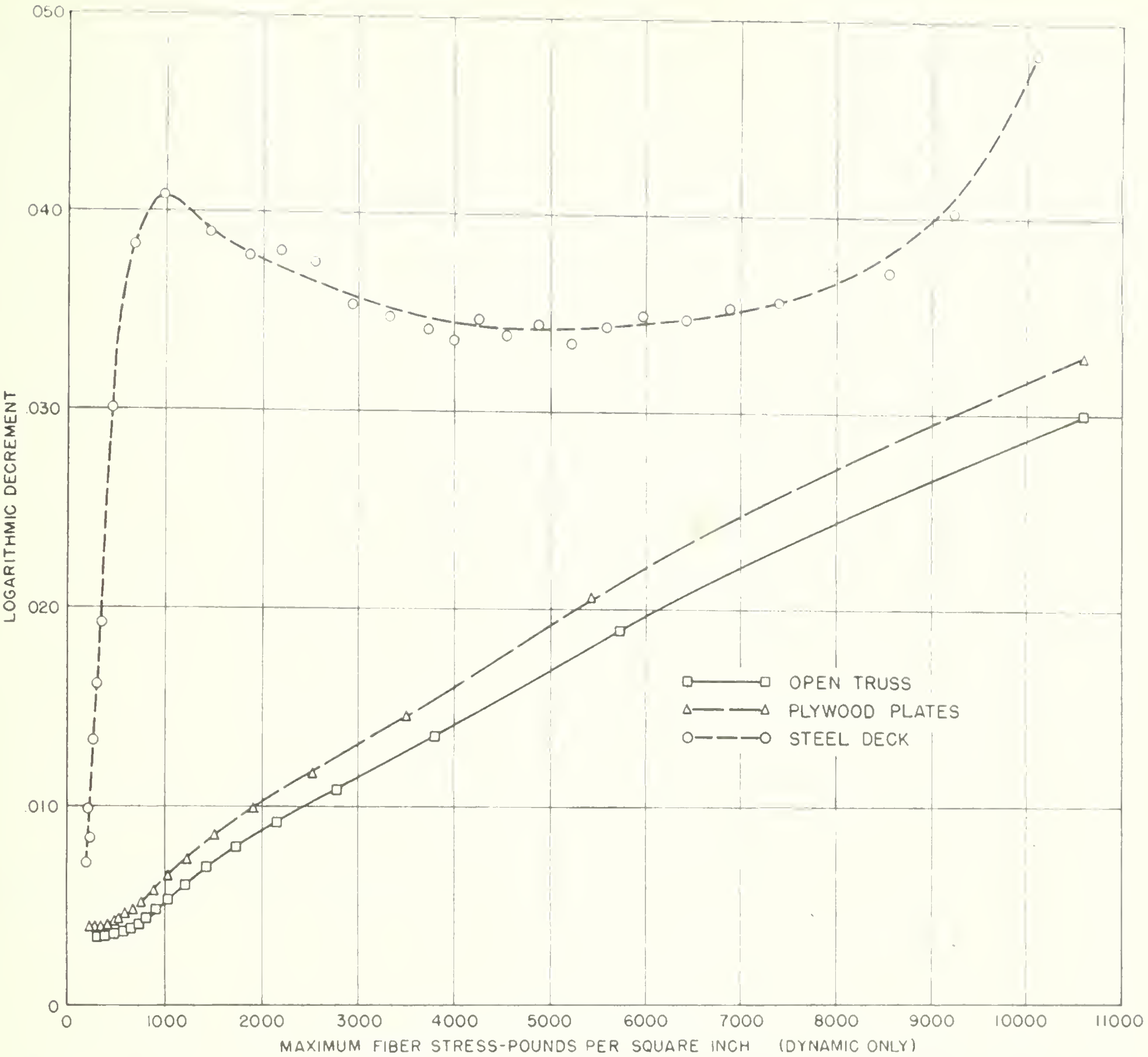

Figure 28.—Variation of logarithmic decrement with fiber stress for three different deck conditions (36-foot, 10-inch truss loaded with 26 weights, each of 34.5 pounds, and bolted with NAS screws at 45 inch-pounds tightening torque).

finished steel plate bolted to the top of the concrete support pedestal. Figure 30 shows a pair of these shoes mounted on the **H**-section. The self-alining bearings made it possible for the sliding plates to remain in flat bearing with the mating surfaces at all times, regardless of any angular motion that might develop in the cross shaft.

A number of tests were made with both types of end bearing with both the truss and **H**-section. For these tests both the 19.5-pound and the 34.5-pound live-load weights were used. Amplitude decay data were obtained and from these logarithmic decrement values were calculated. The data from tests with the 34.5-pound live-load weights are compared with data for other test conditions in figures 31 and 32. In figure 32 the two lower curves show how logarithmic decrement values decreased where the specimens were mounted on ball-bearing wheels and the end bearing friction is small. In contrast, where end bearings of the sliding type are used (without lubrication) and a strong damping force is developed by dry friction, the damping increases with amplitude decrease, rising to a maximum at the 25th oscillation in the case of the truss and at the 30th oscillation in the case of the **H**-beam, after which it decreased very rapidly to a value well below that which obtained at the time of the first oscillation. As in the case of the other tests with external Coulomb friction, these maxima occurred at midspan vibration amplitudes of the order of 0.1 inch at the point where sliding motion of the friction surfaces ceased.

It is of interest to compare the effect on damping of the friction forces developed by the mechanical brakes with those developed by the sliding end bearings. For this purpose the dash line was added to the other data in figure 32. The similarity of the effects of the external resistance derived from dry friction in the two cases is apparent.

Forces Developed at the Sliding Bearings

A study was made of the friction forces developed at the sliding contact of the bronze and steel plates used in most of the friction tests. Information on the character and magnitude of these forces was needed, particularly for the correlation of observed behavior with theory.[5]

To obtain this information an apparatus was designed that would reproduce as closely as possible the conditions present in the friction brake used at the end frame of the truss, and at the same time would provide means for measuring the horizontal forces required to produce sliding of the

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[5] See reference, footnote 1, p. 203.

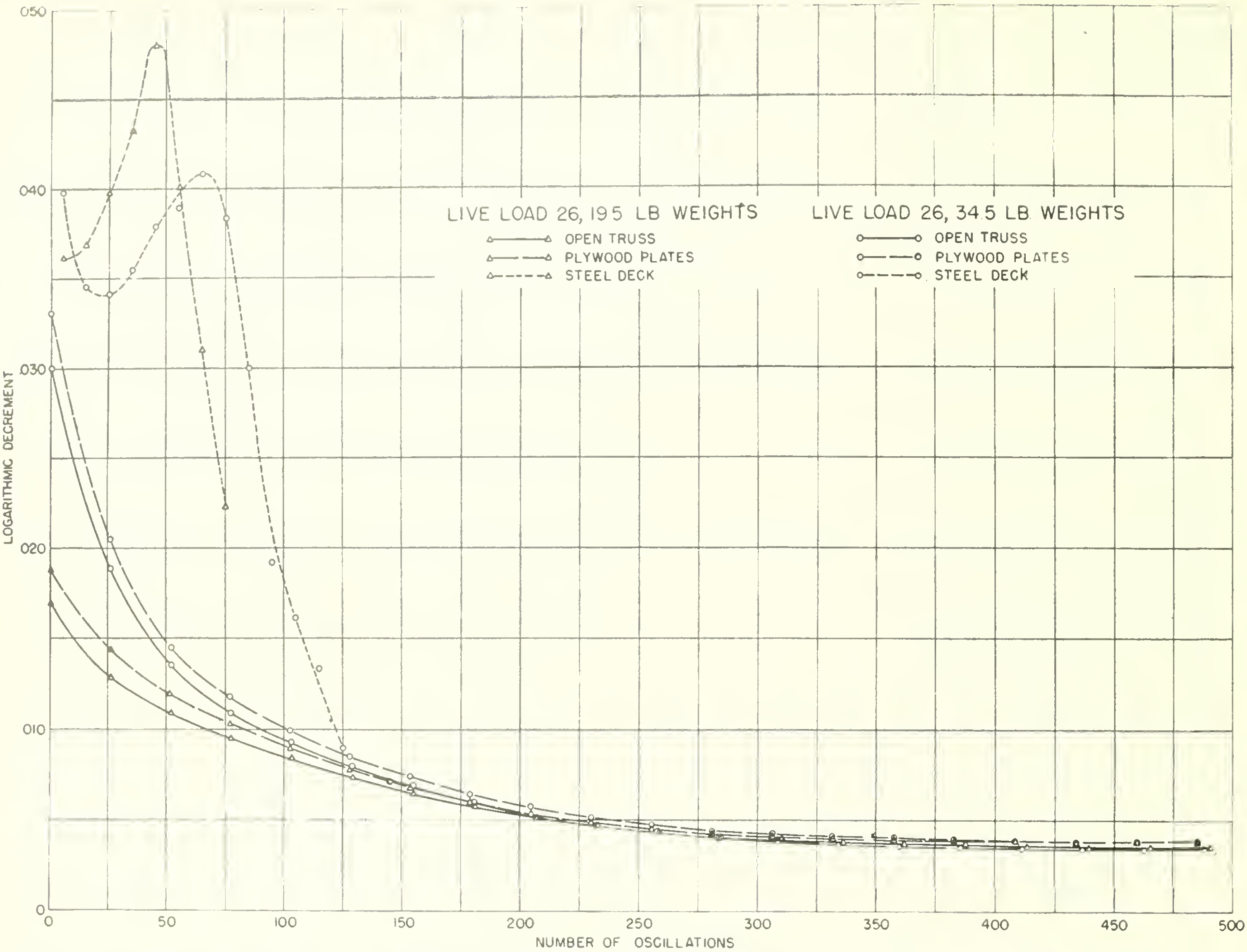

Figure 29.—Effect of variations in deck condition on logarithmic decrement values (36-foot, 10-inch truss bolted with NAS screws at 45 inch-pounds tightening torque).

one friction surface on the other. The apparatus is shown in figure 33. It consists of a motor-driven mechanism that produces a horizontal translatory motion closely approximating in stroke and frequency that of the friction plates in the brake on the truss. This motion is imparted to a small horizontal plate or carriage mounted on small ball-bearing wheels. Included in the linkage is a strain gage dynamometer component to measure continuously the horizontal resisting force created by the friction plates. On the carriage mentioned above the bronze friction plate was fastened, with the steel friction plate superimposed. Vertical pressure between the two was developed by the same lever and weights used in the friction brake on the truss. An oscillograph recorded the entire elastic strain cycle as the dynamometer responded to the resisting forces developed between the two friction plates in the reciprocating motion to which they were subjected.

With this apparatus it was possible also

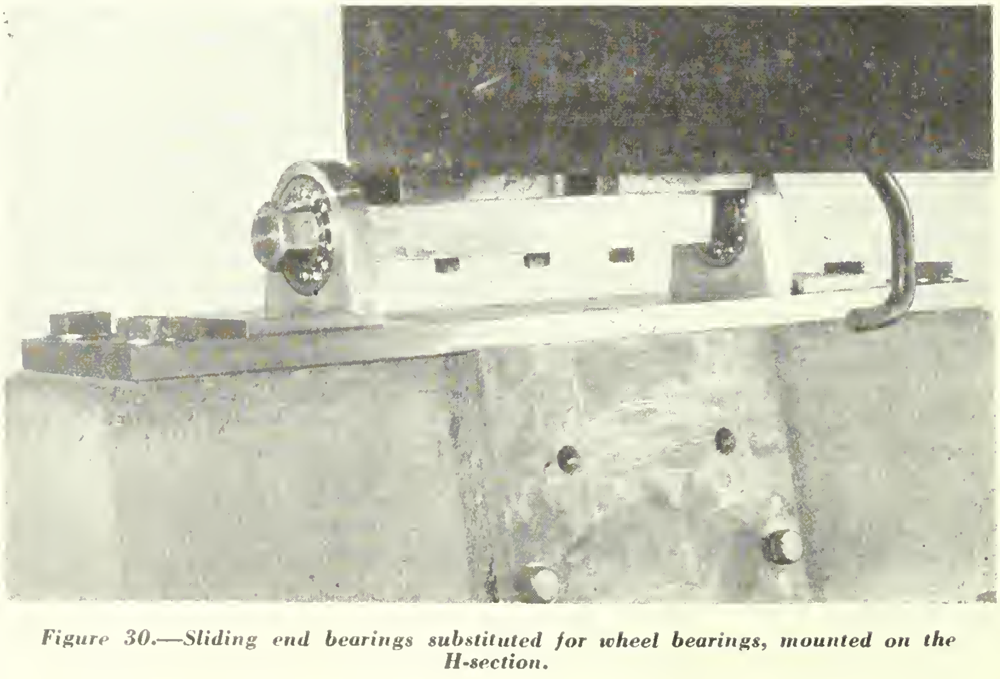

Figure 30.—Sliding end bearings substituted for wheel bearings, mounted on the H-section.

to determine the resistance to horizontal motion offered by the ball-bearing wheels used to support one end of the truss, **H**-section, and rectangular bar in most of the tests. A reproduction of an oscillographic record of tests with the bronze and steel plate combination in an unlubricated condition and with the ball-bearing wheels is shown in figure 34.

From a series of tests with this device it was found that:

1. The force required to cause the steel plate to slide over the bronze plate with the reciprocating motion that has been described varied with time essentially in the form of a square wave, as shown in the lower part of figure 34.
2. Once sliding began, the resistance to motion (or the coefficient of sliding friction) did not vary with the velocity of the motion.
3. The average coefficient of sliding friction for the 2-square-inch steel plate on the bronze plate in an unlubricated condition was found to be 0.131. That with the ball-bearing wheels was too small to measure accurately.

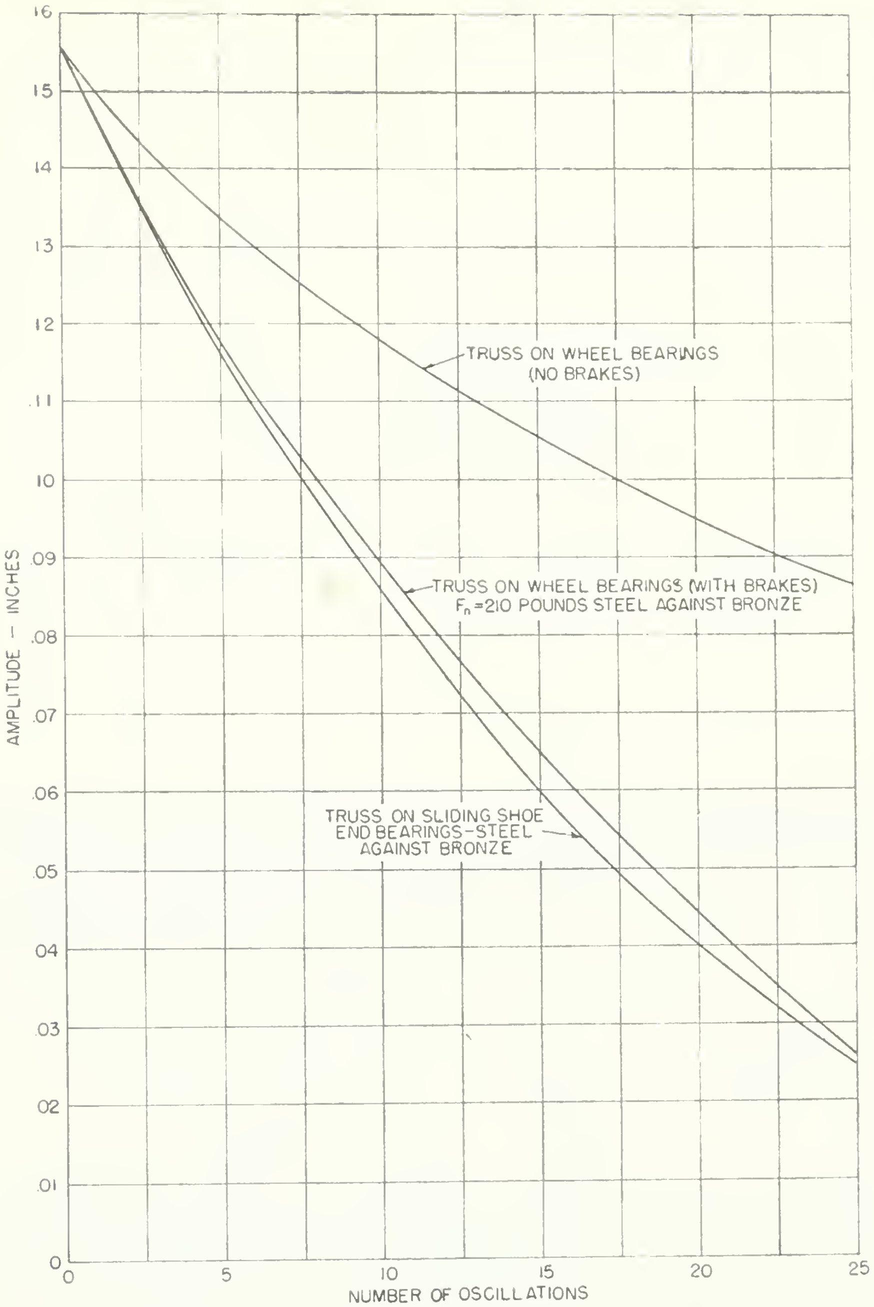

Figure 31.—Comparative effect of brake friction and sliding end bearing friction on amplitude decay.

Tests with Riveted Truss

It appeared desirable before concluding the investigation to make some tests with the truss connections riveted, as a supplement to the tests of the authorized program, in order to afford comparisons between the damping obtained with riveted joints and that found with the several conditions of bolted connections.

The satisfactory riveting of the many connections in the trussed member presented a practical problem of some difficulty. A considerable amount of shop experimentation was necessary before a technique was developed that produced tight rivets, with well formed heads, without distorting the thin metal of the truss members. The rivets used were of soft iron, had an average clearance in the reamed holes of about 0.001 inch before driving, and were driven cold with a small air hammer operating at about 40 p.s.i. pressure. Special tools were made to form the round heads and the length required for the various connections was found by trial. The appearance of typical connections is shown in figure 35 and the overall appearance of the trussed member on its supports is shown in figure 36. The truss was given a spray coat of flat white paint after riveting, with the thought that this might aid in detecting movement in the joints during subsequent testing. The movements proved to be too small to be detected visually, however.

The structural testing activities of the Bureau of Public Roads were moved to a new building in Fairfax County, Va., in 1951. Moving of the testing activities to the new location involved the casting of one new end pedestal, the dismantling and resetting of the end bearings, and other operations that conceivably might cause changes in the conditions of the tests. Since it was desired to make comparisons between data obtained with the riveted truss at the new location and data obtained with the bolted truss at the old location, it was considered desirable to repeat certain tests made with the **H**-section in the early part of the program, to establish the magnitude of any effects that might have developed as a result of moving the test set-up.

For the comparison, tests were made with both the 19.5-pound and the 34.5-pound live-load weights. All details of the original test procedure were repeated as nearly as was possible. Figure 37 shows amplitude decay curves for tests with the 34.5-pound live-load weights at each location, with equations derived to fit each set of experimental data. It will be observed that although the initial amplitudes and those after 500 oscillations are identical there is a slight but definite difference between the shapes of the envelope curves between these points. The equations reflect this difference. Whether the slightly reduced rate of decay at intermediate amplitudes shown in the data obtained in the recent tests is due to reduced friction at the end bearings is not known but it seems probable. These bearings were cleaned and re-alined when the new installation was made. It is significant that, whatever the cause, exactly the same effect was found in the tests with the 19.5-pound live-load weights.

The self-damping of the **H**-section has proved to be relatively small in all of the tests and it is to be expected that the slight

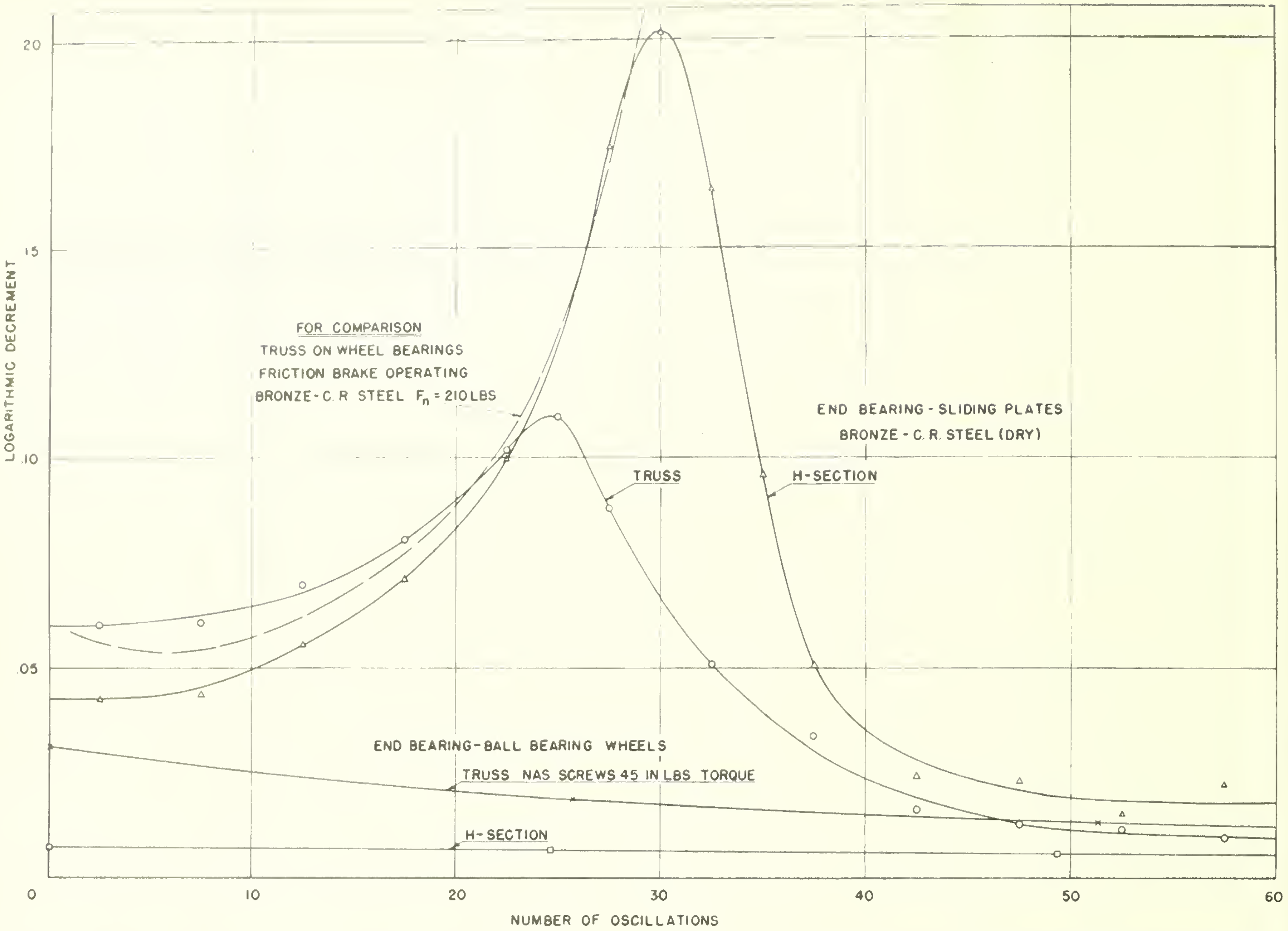

Figure 32.—Effect of type of end bearing on damping of truss and H-section, excited with 26 weights, each of 34.5 pounds. (Damping of truss with friction brake included for comparison.)

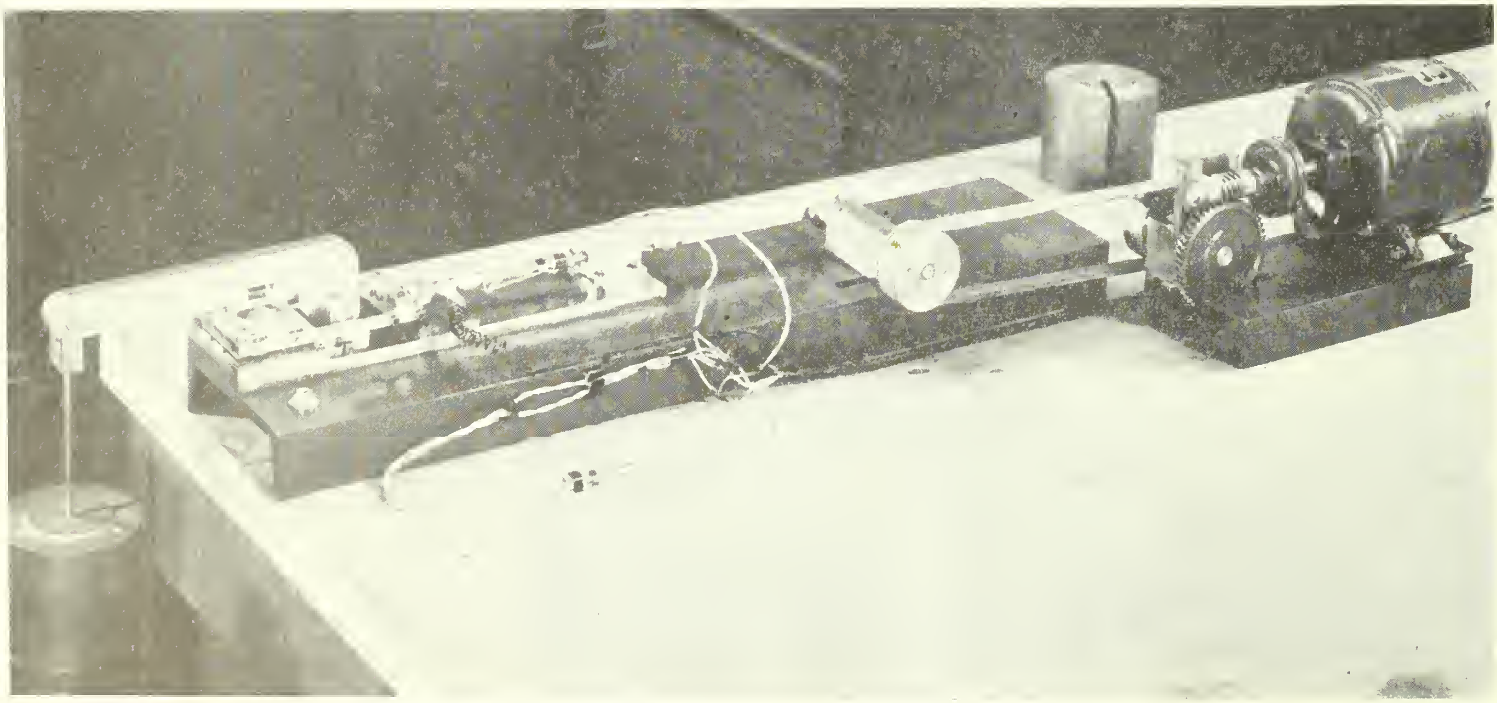

Figure 33.—Apparatus used to evaluate coefficient of friction of surfaces used in brake tests.

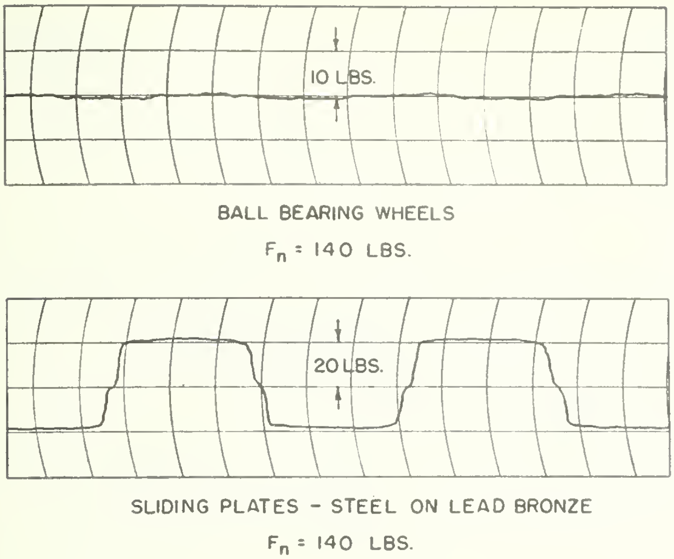

Figure 34.—Relative frictional resistance of ball-bearing wheels and sliding plate during cycles of horizontal motion.

difference in the shape of the two amplitude decay curves of figure 37 would be reflected in a noticeable difference in the logarithmic decrement values at the intermediate and higher amplitudes. That such a difference exists is shown by the decrement-stress relations for the two series of tests as shown in figure 38. It is well to keep in mind the generally low order of magnitude of the decrement values for the **H**-section. Differences such as are shown in figure 38 are of relatively small significance when comparing data from tests with the truss in either the bolted or the riveted condition.

Comparison of test data

Following the tests with the **H**-section, the riveted truss was set up on the 36-foot, 10-inch span and tested with both the 19.5-pound and the 34.5-pound live-load weights, using exactly the same testing procedure that had been employed in the earlier tests with the bolted truss. Since the tests with the 19.5-pound weights developed no information that was not shown by the tests with the 34.5-pound weights, only the data from the latter will be included in this report.

It was found in the early tests with the truss bolted with machine screws at 35 inch-pounds of torque that a permanent sag developed in the member as a result of slight movements in the many joints. For the bolted truss the magnitude of this sag at midspan was 1.09 inches. Measurements made during the testing of the riveted truss showed the sag at midspan after testing to be 0.04 inch or about 4 percent that of the bolted truss. Moment of inertia values computed from the measured load-deflection characteristics of the truss were 20.7 inches^4 when bolted and 20.6 inches^4 when riveted. These data indicate less movement in the joints of the riveted truss but little or no change in its stiffness.

Logarithmic decrement values computed from amplitude decay data for the riveted truss are shown in figure 39, plotted against dynamic stress values obtained by means of strain measurements. Similar curves for the truss bolted up with both the No. 10–24 machine screws and the NAS screws, as well as that from tests with the **H**-section, are shown in the same figure for comparison.

It is indicated by the comparisons available in this figure that while reduced movements in the joints of the riveted truss did lower appreciably the self-damping of the member below that of the bolted truss, particularly at higher amplitudes (and stresses), it still showed much greater self-damping at all amplitudes than was found for the comparable solid member (**H**-section).

A test of a riveted chord splice in tension failed by rupture of both of the splice plates in the grip. No failure occurred in the riveted splice although the computed unit stress in the chord angles was about 68,100 p.s.i. It will be recalled that the tension test of a chord splice drawn up with the NAS screws at 15 inch-pounds torque developed failure by rupture of the chord angles through the first bolt hole at a computed stress in the angles of about 76,600 p.s.i. Data from the test of the riveted splice are included in table 3.

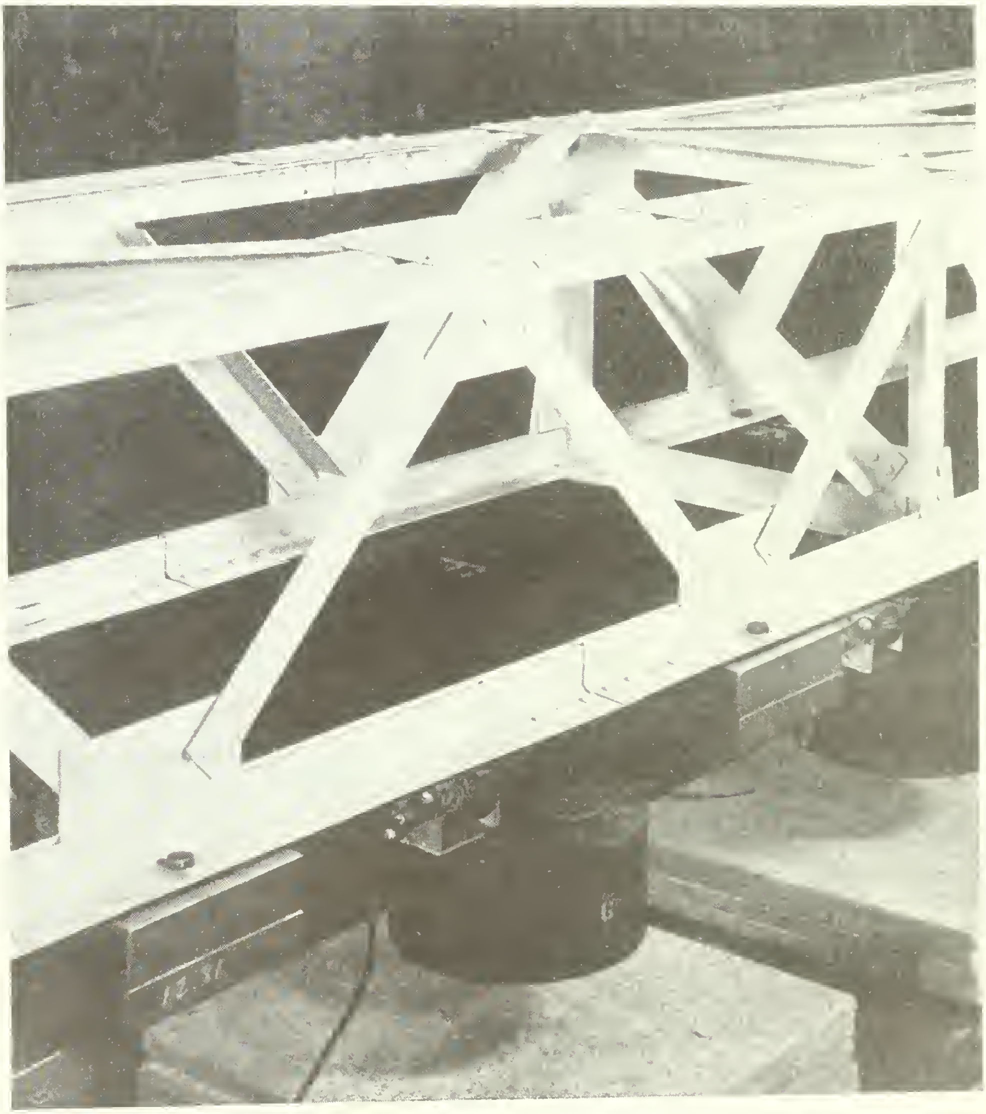

Figure 35.—Appearance of riveted connections after applying spray coat of flat white paint.

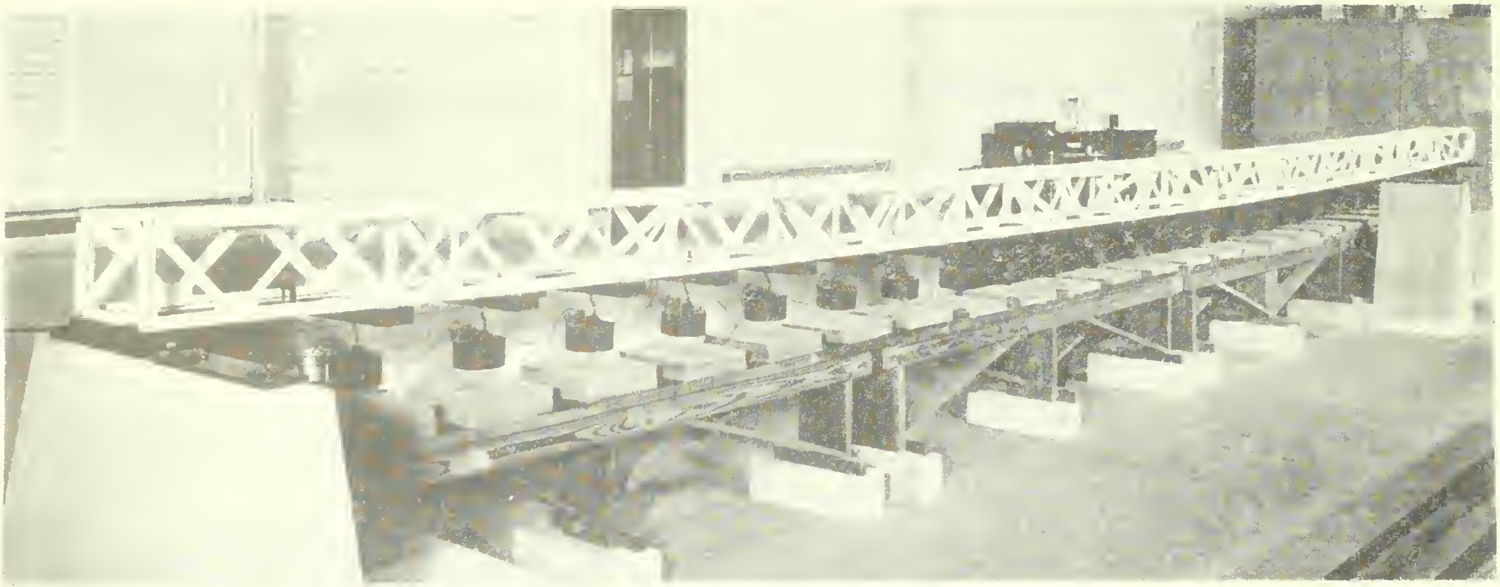

Figure 36.—General appearance of riveted truss at new test site.

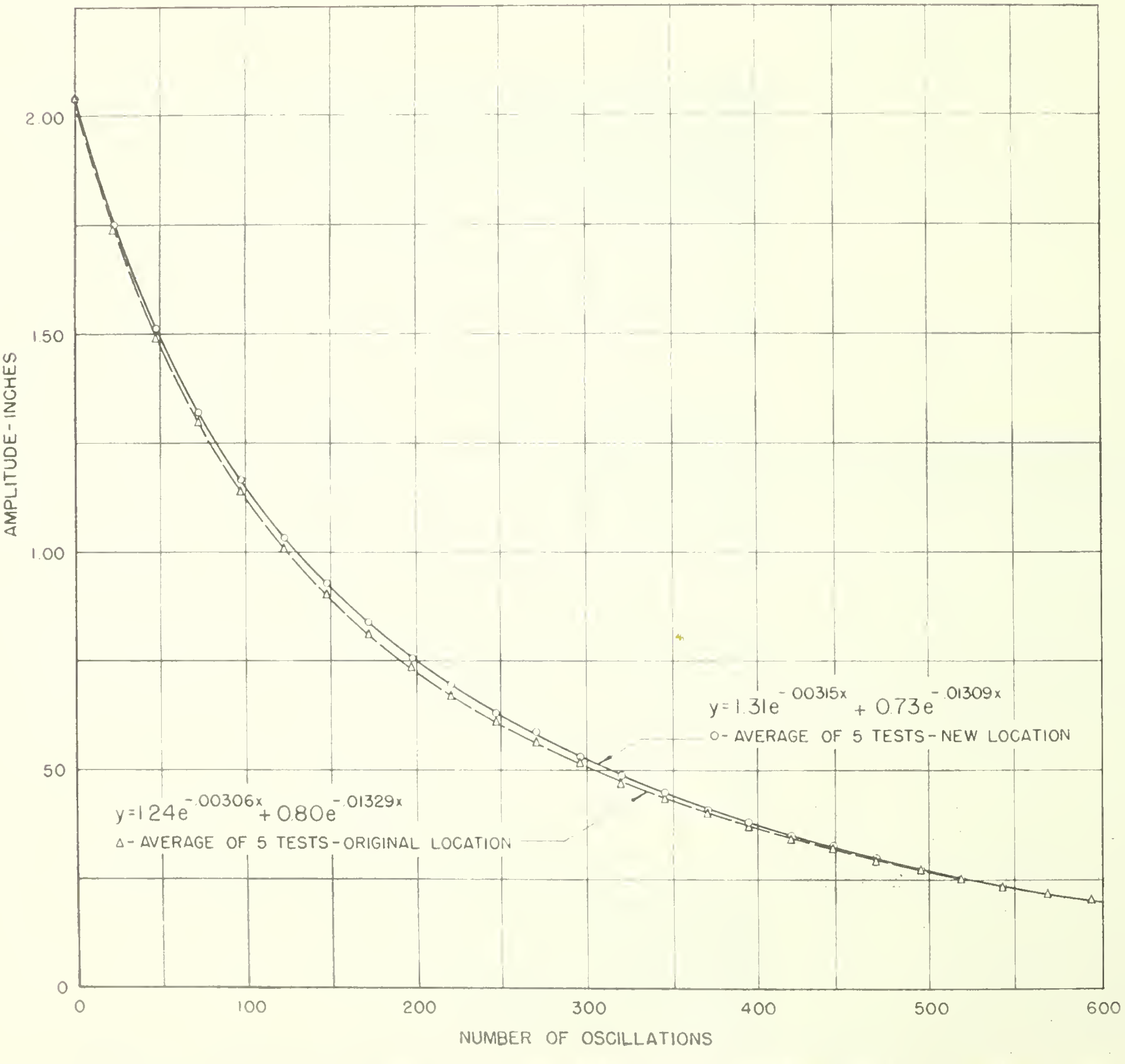

Figure 37.—Comparison of amplitude decay curves obtained with the H-section at the two test sites.

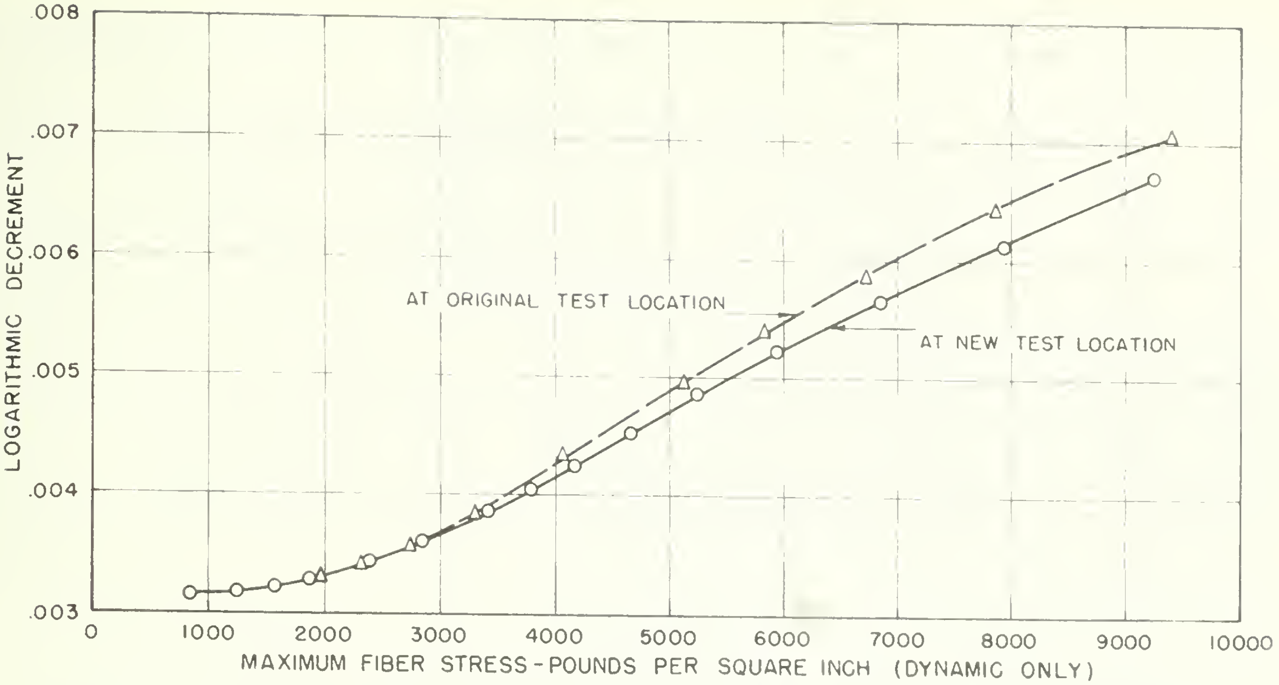

Figure 38.—Comparative logarithmic decrement data obtained with the H-section at the two test sites.

Comparative Measurements of Damping

It was discovered at the time of the first tests of the original program that the vertical oscillation of the test specimens was responsive to very small amounts of friction in the recording apparatus at midspan, so care was taken to reduce the friction of the stylus in the drum recording equipment to a minimum. The use of a method that yielded direct recordings of amplitude at midspan had considerable appeal since there could be no doubt of the extent of the motion that it was desired to measure. However, it was realized that some stylus friction was always present and the effect of this could not be readily evaluated.

Later, when equipment became available that permitted measurements of oscillation amplitude without physical contact with the specimen, it was felt that this might be a better method for obtaining amplitude decay data. As stated earlier, the magnetic reluctance pickup, having been designed for another purpose, had an effective range of motion that was too small for measurements at midspan so it was necessary to install it near one reaction and to establish by test the proportionality of vertical amplitudes there with those at midspan. This was done by simultaneous measurements at midspan with the smoked paper on the drum and measurements near the rocker reaction with the magnetic cell. A good general proportionality was found to exist. Because of the amplification required for recordings with the magnetic cell, the element of instrumental performance was present in these data. The installation of this unit at the new test site is shown in figure 40.

The question of determining damping characteristics through the analysis of continuous recordings of strain was discussed early in the program. At that time suitable equipment was not available and experiments made with the equipment at hand, while encouraging, showed the need for better instrumentation if the subject were to be investigated.

Such equipment, acquired for other purposes, was available at the time of the tests with the riveted truss and for this reason some comparative tests were made, both with the **H**-section and with the riveted truss. In these tests simultaneous measurements were made of midspan deflection with smoked drum recording, midspan strain with amplification and oscillograph recording, and of deflection amplitudes at a point about 14 inches from one end reaction using a magnetic reluctance pickup and oscillograph recording. From each of these records amplitude decay curves were obtained and logarithmic decrement values computed. The computed logarithmic decrement values plotted against a number-of-oscillation (or time) scale are shown for both specimens and for the three types of recorded data in figure 41. Considering the variables that are involved in the recording and reducing of the data obtained in this study, it is considered that a rather good correlation was obtained. Curiously, the lowest decrement values were consistently those derived from data obtained with the direct recording by means of the stylus on the smoked drum. This suggests the possibility that characteristics of the electrical amplification circuits employed in the other two recording systems may have been responsible for such dispersion as is shown.

Some Comparisons of Experimental Data with Theory

In the paper published previously[6] there was a discussion of the motions which occurred at the friction surfaces where the self-damping of the truss was augmented by dry friction externally applied. This discussion was a part of the theoretical development preliminary to obtaining comparisons between theoretical and observed relations of vibration amplitude versus damping capacity as measured by values of logarithmic decrement. Subsequent work done, as one of the supplemental investigations previously mentioned, yielded information of value regarding this relation.[7]

Measurements were made of the translatory and rotational movements at the two ends of the **H**-section as caused by the release of the 26 weights of 34.5 pounds each, symmetrically arranged at uniform spacing. Under the conditions of these tests the tendency for translatory movement at the end supports is the result of two separate deformations, both of which vary as the beam vibrates. In the first place, the deflection of a beam that is simply supported is accompanied by a shortening of its upper fibers under compression and a lengthening of its lower fibers under tension. This is reflected in a rotational motion of the transverse sections immediately over the supports. In the present case the beam had considerable depth and the supports were below the neutral axis so that the rotation

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[6] See footnote 1, p. 203.

[7] Joseph H. Appleton, Assistant Structural Engineer, participated in the conduct of this portion of the program of supplementary tests. His assistance is gratefully acknowledged.

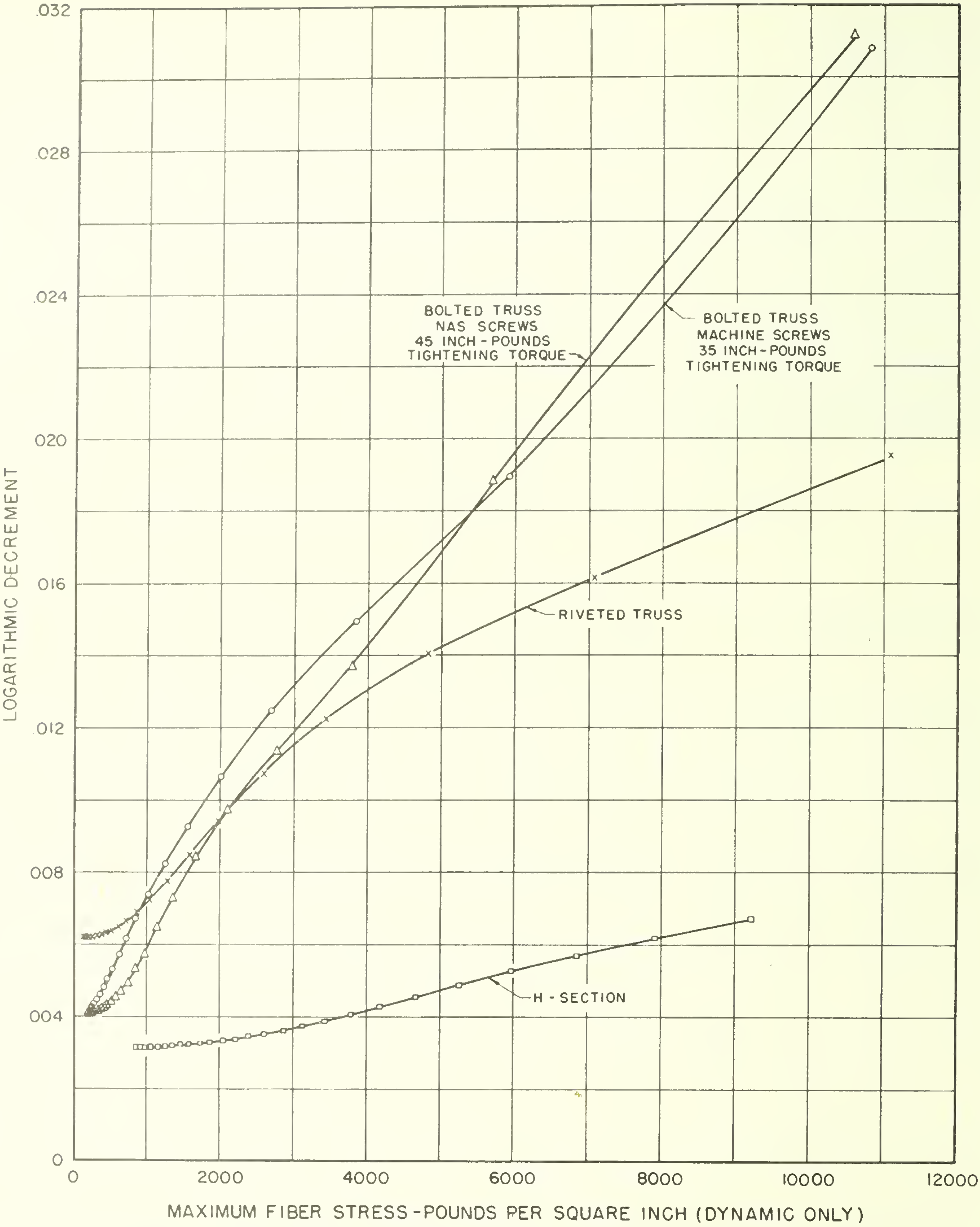

Figure 39.—Variation of logarithmic decrement with fiber stress for the truss and H-section (36-foot, 10-inch span with 26 weights, each of 34.5 pounds).

of the end sections developed a tendency for translatory motion at the end supports. Secondly, any change in the vertical deflection of the beam is accompanied by a change in the length of the chord between the two ends of the neutral axis. This also causes a tendency for translation at the end supports. Since at the rocker bearing there was restraint against longitudinal motion, all of the translatory motion at the supports had to take place at the wheel bearing end of the beam.

The theoretical movements, both rotational and translatory, were computed for the particular conditions of these tests and were compared with motions as measured with dial micrometers. The comparison is shown in table 5.

Having obtained a satisfactory correlation for the **H**-section, a study was made of the movements that would occur at the ends of the bolted truss, particularly at the position of the friction surfaces used in the tests with the friction brakes. Since the damping capacity depends directly upon the extent of the movement at these surfaces, it was desirable that a rather precise comparison be made. This led to a study of the effect of the sag of the truss, which has been mentioned before, on the motions that would occur if the sag were not present. The magnitudes of the movements were computed, both with and without a correction for sag, and from these logarithmic decrement values for various amplitudes were obtained, as shown in figure 42. The results are compared with a similar relation obtained from measured amplitude and logarithmic decrement values.

It is apparent that the sag present in the bolted truss was sufficient to have a measurable effect on the computed value of damping capacity. It is indicated also that when a correction for sag is introduced there is somewhat better agreement between the computed and observed data shown in this graph. Furthermore, this figure shows a good correlation between the observed behavior of the truss with strong Coulomb damping and that predicted by theory for the same conditions.

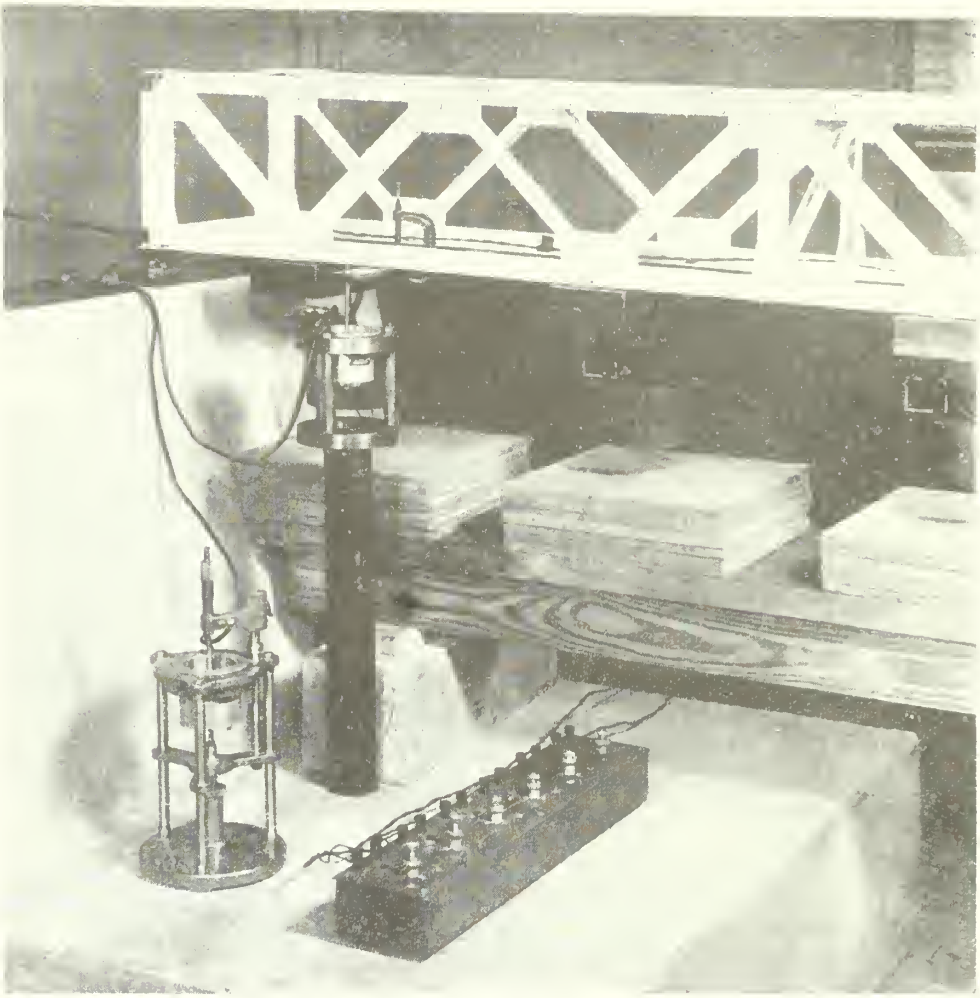

Figure 40.—Magnetic reluctance pickup device located about 14 inches from the rocker end bearing on the riveted truss.

Table 5.—Comparison of computed and observed rotational and translatory movements at end supports

	Computed movement	Observed movement
End rotation:		
Rocker end . . . radians	0.0149	0.0147
Wheel-bearing end . . . do	0.0149	0.0148
Translation, resultant outward, at axis of wheel bearing . . . inch	0.053	0.054
Translation, resultant inward, at end of neutral axis:		
Rocker end . . . do	0.061	0.061
Wheel-bearing end . . . do	0.008	0.007

(Figs. 41 and 42 are on the following pages.)

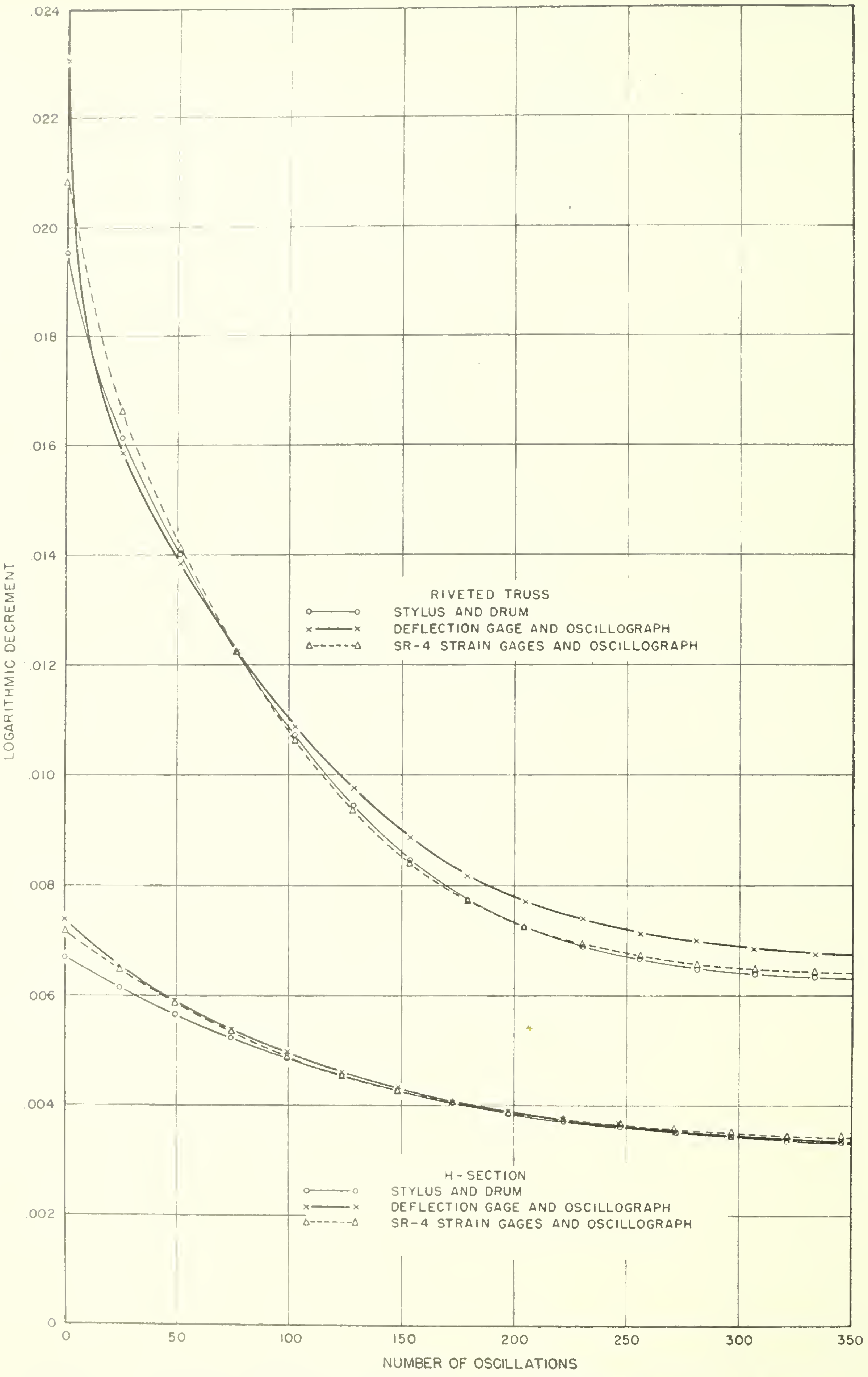

Figure 41.—Comparative logarithmic decrement values obtained with three methods of measurement.

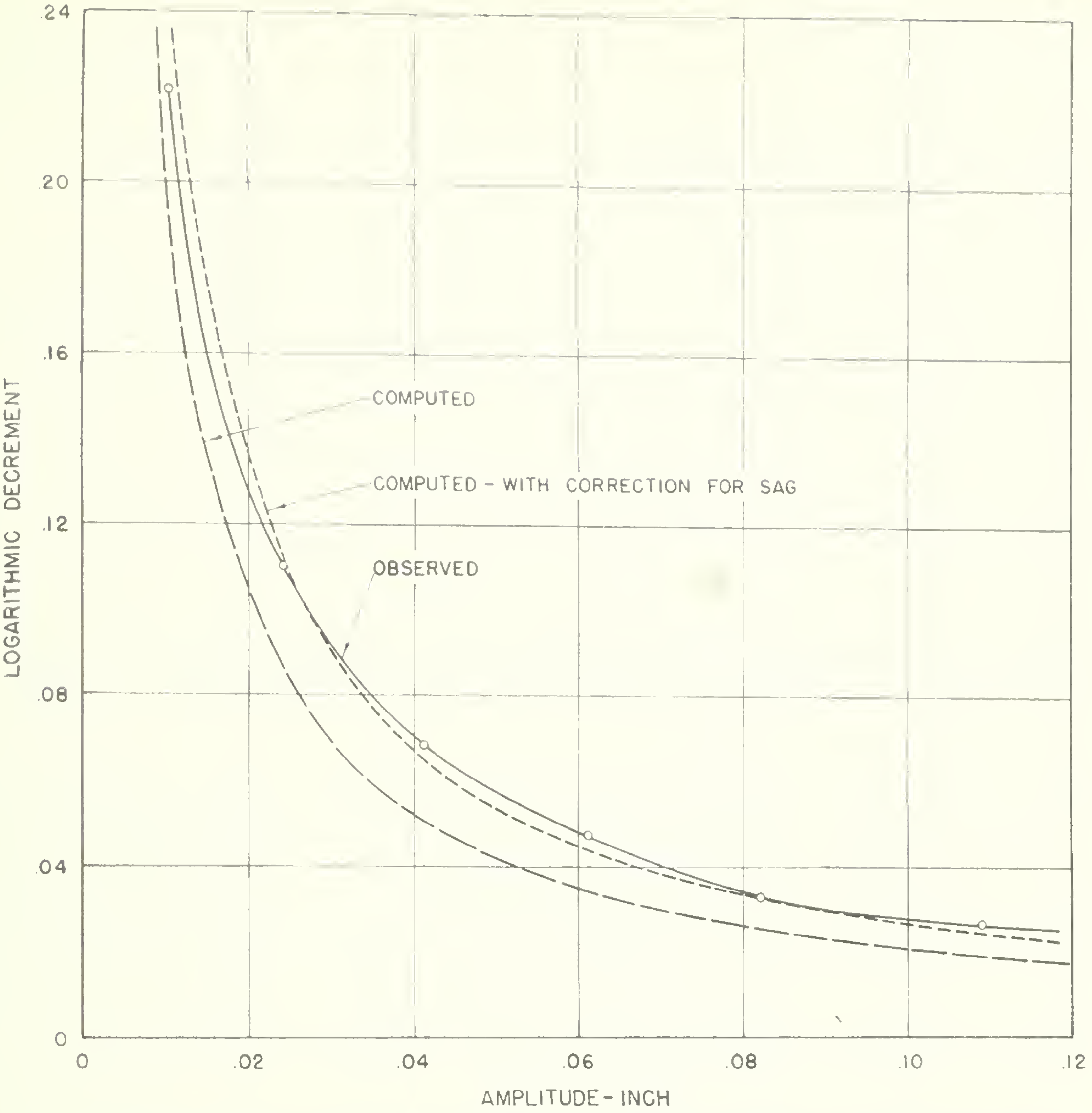

Figure 12.—Effect of correction for sag of bolted truss on the correlation between observed and computed logarithmic decrement-amplitude relations (friction brake tests, $F_n = 210$ *pounds).*

Results of Physical Tests of Road-Building Aggregate: a new bulletin

The Bureau of Public Roads has recently published a new bulletin, *Results of Physical Tests of Road-Building Aggregate*, which reports the results of tests performed by the Bureau's laboratory on more than 9,700 samples of ledge rock, crushed stone, gravel, and blast-furnace or smelter slag, and more than 3,400 samples of natural or manufactured fine aggregate, received from sources all over the United States. The bulletin is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $1.00 a copy.

Since the value of material for use as aggregate in road construction depends largely on the extent to which it will resist the destructive forces of traffic and weather, and on its ability to exist in harmony with other materials with which it is used, the test data reported in this bulletin serve, to some extent, as measures of these qualities. Thus, for engineers and contractors, the bulletin will be a useful guide on the availability and suitability for highway construction of local coarse and fine aggregates.

A brief introductory text in the bulletin reports the nature of the tests performed and describes the rocks and minerals mentioned in the tables. Results of tests on coarse and fine aggregates are tabulated separately, and are arranged in alphabetical order by State, county, and nearest town. The type of source is indicated as commercial, local, or prospective.

For both coarse and fine aggregates, the bulletin reports the name or lithological composition of the material, and results of tests of bulk gravity, absorption, and soundness. In addition, for coarse aggregate, test results of abrasive loss, crushing strength, hardness, toughness, and weight of compacted aggregate are reported; for fine aggregate, test results of grading, fineness modulus organic matter content, and mortar strength are shown. All tests were not performed on all samples, however.

The tests reported in *Results of Physical Tests of Road-Building Aggregate* cover the work of the Bureau of Public Roads in this field over the 56-year period from 1895 to January 1, 1951. The bulletin supersedes and supplements U. S. Department of Agriculture Miscellaneous Publication No. 76, *The Results of Physical Tests of Road-Building Rock*, published in 1930 and long out of print.

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

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Act III.—Uniform Motor-Vehicle Civil Liability Act (1944). 10 cents.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act. 15 cents. (revised 1952)

Act V.—Uniform Act Regulating Traffic on Highways. 20 cents. (revised 1952)

Model Traffic Ordinance. 20 cents. (revised 1952)

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.

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MISCELLANEOUS PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.
Construction of Private Driveways, No. 272MP (1937). 10 cents.
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Highway Bridge Location, No. 1486D (1927). 15 cents.
Highway Capacity Manual (1950). 65 cents.
Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.
Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):

1945, 35 cents. 1948, 65 cents.
1946, 50 cents. 1949, 55 cents.
1947, 45 cents. 1951, 60 cents.

Highway Statistics, Summary to 1945. 40 cents.
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Transition Curves for Highways (1940). $1.50.

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Bibliography on Automobile Parking in the United States (1946).

Bibliography on Highway Lighting (1937).

Bibliography on Highway Safety (1938).

Bibliography on Land Acquisition for Public Roads (1947).

Bibliography on Roadside Control (1949).

Express Highways in the United States: a Bibliography (1945).

Indexes to PUBLIC ROADS, volumes 17–19 and 23.

Title Sheets for PUBLIC ROADS, volumes 24, 25, and 26.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUGUST 31, 1953

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$5,550	$19,288	$9,895	305.6	$11,062	$5,511	204.2	$37,922	$19,468	474.8	$68,272	$34,874	984.6
Arizona	1,273	3,404	2,379	76.6	1,322	918	29.7	6,367	4,529	89.0	11,093	7,826	195.3
Arkansas	4,073	12,889	6,765	363.5	3,177	1,592	105.6	11,579	5,714	284.1	27,645	14,071	753.2
California	2,938	12,975	4,729	64.8	6,834	3,674	48.1	105,084	52,207	317.3	124,893	60,610	430.2
Colorado	2,442	6,431	3,642	150.7	3,633	2,052	63.8	16,796	9,086	170.0	26,860	14,780	384.5
Connecticut	6,959	1,902	1,003	3.6	475	235	.3	12,504	6,150	35.5	14,881	7,388	39.4
Delaware	2,610	605	305	.8	1,179	589	7.6	3,547	1,766	14.5	5,331	2,660	22.9
Florida	4,682	16,459	8,359	255.2	7,035	3,809	127.6	17,639	8,894	239.5	41,133	21,062	622.3
Georgia	7,462	17,137	8,800	440.2	8,483	4,018	152.0	33,997	16,259	487.5	59,617	29,077	1,079.7
Idaho	2,634	9,584	6,003	169.4	3,343	2,097	66.5	11,833	7,513	242.5	24,760	15,613	478.4
Illinois	9,006	30,775	16,570	144.0	19,704	10,534	117.4	70,029	36,210	594.7	120,508	63,314	856.1
Indiana	10,514	36,255	18,882	161.6	6,427	3,221	50:0	27,253	14,350	148.8	69,935	36,453	360.4
Iowa	2,779	14,778	8,105	368.5	6,651	4,138	272.7	18,024	9,076	827.7	39,453	21,319	1,468.9
Kansas	4,445	9,315	4,654	845.2	7,350	3,693	396.7	17,993	8,654	794.5	34,658	17,001	2,036.4
Kentucky	3,813	11,968	6,188	133.4	5,218	2,609	101.4	23,112	12,010	300.6	40,298	20,807	535.4
Louisiana	1,735	21,097	10,577	139.6	3,446	1,721	17.7	26,524	12,829	137.6	51,067	25,127	294.9
Maine	958	7,834	3,931	60.5	2,591	1,278	27.6	13,300	6,434	81.2	23,725	11,643	169.3
Maryland	8,210	8,117	4,232	70.5	3,743	1,717	27.3	6,168	3,567	35.4	18,028	9,516	133.2
Massachusetts	3,268	16,293	8,241	24.6	3,074	1,525	4.1	40,143	19,133	34.6	59,510	28,899	63.3
Michigan	6,746	23,418	12,299	415.2	12,068	6,060	261.0	53,131	22,903	294.4	88,617	41,262	970.6
Minnesota	4,155	11,981	6,469	920.4	3,691	2,065	280.3	21,268	11,338	706.8	36,940	19,872	1,907.5
Mississippi	1,251	15,941	8,064	443.2	3,863	1,822	161.6	22,009	11,242	637.5	41,813	21,128	1,242.3
Missouri	3,952	15,850	8,193	705.5	4,099	2,058	111.2	56,415	27,863	528.6	76,364	38,114	1,345.3
Montana	7,564	10,935	6,732	196.2	4,545	2,809	98.2	16,340	9,943	279.3	31,820	19,484	573.7
Nebraska	13,147	13,110	7,008	541.4	2,183	1,346	47.7	11,810	6,545	257.7	27,103	14,899	846.8
Nevada	4,899	3,877	3,246	71.2	438	367	7.8	4,752	3,940	104.2	9,067	7,553	183.2
New Hampshire	2,592	2,627	1,307	13.1	721	360	5.5	6,236	3,237	36.4	9,584	4,904	55.0
New Jersey	3,332	7,782	3,891	61.6	7,819	3,722	8.6	26,506	12,686	25.0	42,107	20,299	95.2
New Mexico	1,132	2,760	1,727	54.1	3,610	2,288	93.0	6,828	4,329	153.4	13,198	8,344	300.5
New York	23,536	79,128	40,598	114.2	28,798	14,681	125.4	148,271	68,438	435.9	256,197	123,717	675.5
North Carolina	6,254	19,405	9,422	365.9	5,764	2,726	122.1	28,359	13,587	497.6	53,528	25,735	985.6
North Dakota	2,532	4,908	2,468	823.9	2,470	1,235	352.2	10,920	5,638	781.6	18,298	9,341	1,957.7
Ohio	7,686	17,161	7,899	116.6	9,816	4,995	36.2	95,299	47,120	149.6	122,276	60,014	302.4
Oklahoma	8,791	11,197	6,263	156.9	4,200	2,216	91.0	16,479	8,665	204.0	31,876	17,144	451.9
Oregon	3,288	809	433	16.2	1,141	677	46.8	14,112	8,541	202.9	16,062	9,651	265.9
Pennsylvania	5,168	34,211	15,578	32.2	28,884	13,770	66.8	91,052	45,289	236.3	154,147	74,637	335.3
Rhode Island	1,944	3,277	1,638	31.9	161	80		9,831	4,912	22.2	13,269	6,630	54.1
South Carolina	2,881	11,504	6,310	307.8	2,313	1,162	120.7	16,613	8,415	318.7	30,430	15,887	747.2
South Dakota	688	8,279	4,784	497.6	4,024	2,250	227.8	8,316	4,901	429.4	20,619	11,935	1,154.8
Tennessee	4,455	10,643	5,301	363.4	10,633	6,007	323.6	32,201	14,151	238.9	53,477	25,459	925.9
Texas	11,600	6,607	3,509	125.7	12,785	6,746	385.9	59,893	33,306	820.9	79,285	43,561	1,332.5
Utah	492	3,026	2,314	47.1	2,513	1,870	50.0	13,268	10,051	174.1	18,807	14,235	271.2
Vermont	614	4,081	2,160	41.8	418	286	1.6	9,083	4,522	62.4	13,582	6,968	105.8
Virginia	965	14,018	5,990	202.8	9,009	4,360	101.9	32,876	15,960	241.6	55,903	26,310	546.3
Washington	1,047	9,937	5,146	130.3	3,017	1,545	86.7	17,203	9,206	135.6	30,157	15,897	352.6
West Virginia	3,872	8,400	4,227	44.0	4,815	2,431	29.2	15,649	7,841	112.8	28,864	14,499	186.0
Wisconsin	3,469	11,674	6,268	190.1	4,383	2,341	119.4	36,792	18,677	415.3	52,849	27,286	724.8
Wyoming	624	1,960	1,242	48.0	1,725	1,116	59.9	8,636	5,696	167.6	12,321	8,054	275.5
Hawaii	1,027	3,248	1,590	7.8	161	81	.6	11,667	5,625	18.5	15,076	7,296	26.9
District of Columbia	73	7,788	3,654	6.0	4,653	2,131	.2	10,094	4,742	3.5	22,535	10,527	9.7
Puerto Rico	3,643	10,385	4,761	51.2	1,429	700	6.7	14,755	7,045	47.9	26,569	12,506	105.8
TOTAL	235,155	647,033	333,751	10,921.6	290,896	151,234	5,249.9	1,426,478	720,203	14,050.4	2,364,407	1,205,188	30,221.9

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DECEMBER 1953

Public Roads

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Published Bimonthly

Edgar A. Stromberg, ***Editor***

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The printing of this publication has been approved by the Director of the Bureau of the Budget, January 5, 1952.

Contents of this publicotion moy be reprinted. Mention of source is requested.

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretory

BUREAU OF PUBLIC ROADS
FRANCIS V. du PONT, Commissioner

Trends in Traffic Volumes, Vehicle Types, and Weights

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
THOMAS B. DIMMICK,
Head, Current Data Analysis Unit

Total travel on all rural roads in 1952 broke all records, exceeding the 1951 previous high by more than 6 percent. On the 361,000 miles of main rural roads in the United States, travel in 1952 was a little more than 203 billion vehicle-miles, of which 77 percent was by passenger cars, 1 percent by buses, and 22 percent by freight-carrying vehicles.

Trucks and combinations hauled 3 percent more ton-mileage of freight on main rural roads in 1952 than in 1951. Single-unit truck travel was 11 percent higher than in 1951 while that of combinations increased only 2 percent. The average carried load for all trucks and combinations in 1952 decreased slightly.

In 1952 over 5 percent of all trucks and combinations exceeded a State legal weight limit, and more than 15 percent of the combinations were illegally overloaded in some particular. In comparison with 1951 the percentage of overweight vehicles for 1952 remained the same in the West South Central States, and increased in all other regions except Middle Atlantic, East North Central, and West North Central States.

RURAL MOTOR VEHICLE TRAVEL broke all previous records in 1952 for the seventh consecutive year. The estimated 1952 traffic on all rural roads was about 6 percent above the 1951 total, 18 percent above 1950, 28 percent above 1949, 40 percent above 1948, 49 percent above 1947, and 63 percent above 1946. Data collected from January through August in 1953 indicate that travel on all rural roads in 1953 will continue the same general trend and will be almost 5 percent higher than in 1952.

The variation in average daily travel on rural roads by months in the three main geographic divisions[1] and in the United States as a whole is illustrated in figure 1 for the years 1951, 1952, and the first 8 months of 1953. Travel in each month of these years in the Central region and in the United States as a whole was well above that of the corresponding month of the earlier year. The Eastern regions showed only a slight gain in September 1952 compared to 1951, but a fairly steady gain in all other months. The Western regions showed only slight gains in travel in January, February, March, October, and November of 1952 compared to that in the same months of the previous year and fairly steady gains in all other months.

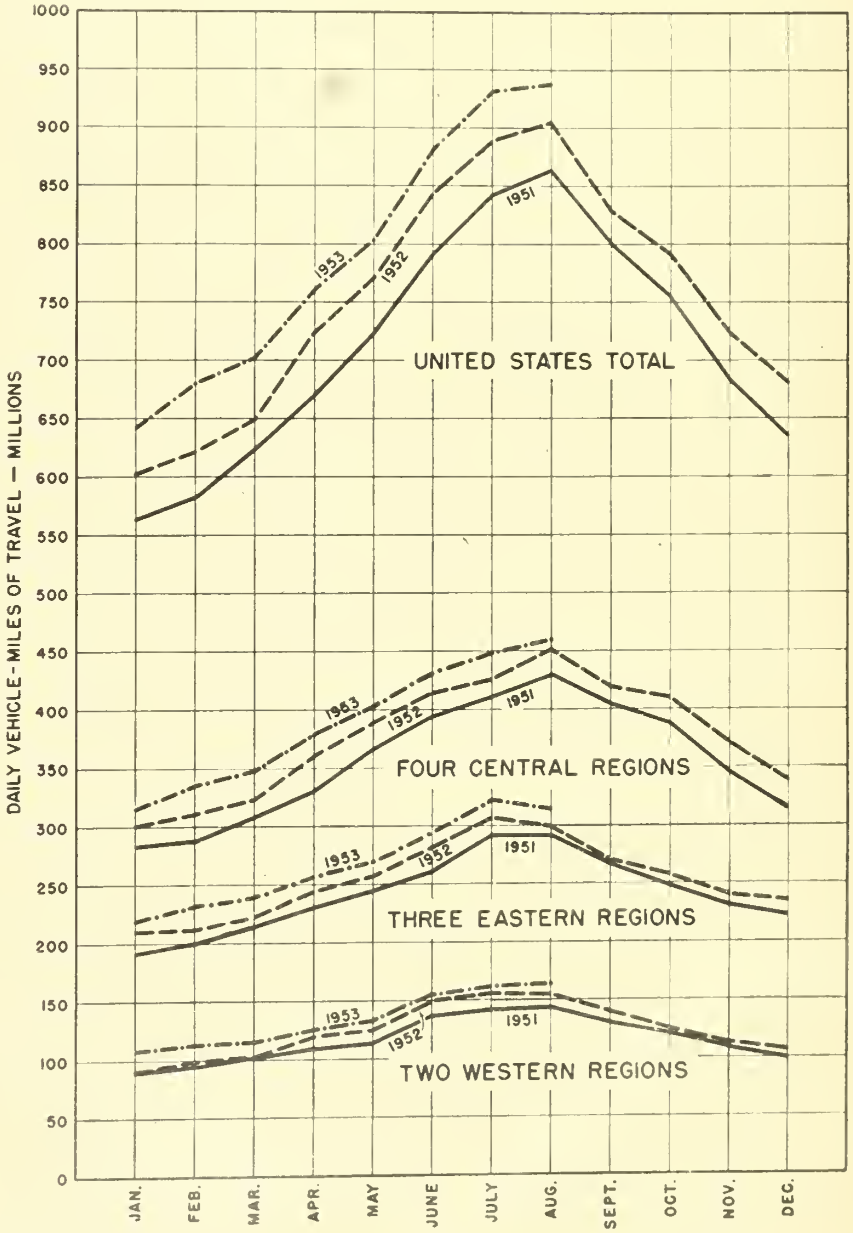

Figure 1.—Travel on all rural roads in 1951, 1952, and in the first 8 months of 1953.

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[1] The States comprising each census region and the regions comprising each geographic division are indicated in table 1.

Approximately the same rate of increase in 1953 over 1952 is indicated by data collected in the first portion of the current year. The partial 1953 information indicated an increase of over 5 percent in the Eastern region and slightly less than 5 percent in the Central and Western States. The largest indicated increase over 1952 in any census region was 10 percent in New England and the smallest was 2 percent in the West South Central region.

Summer travel constituted a smaller portion of the annual travel in 1952 than in any recent year. In the last two prewar years (1940 and 1941), the average daily traffic in July and August was 23 percent above the average traffic of the year. Not until 1949 did the summer travel reach the prewar ratio. In 1950 the average daily summer travel was over 24 percent above the annual average daily amount, but this figure slumped to 22 percent in the 1951 summer season and to 21 percent in 1952.

Figure 2, showing travel on all rural roads by 12-month periods ending each month (moving average) and as a percentage of that in the calendar year 1941, gives an accurate picture of the effect of wartime restrictions and the steady traffic growth that has occurred since the end of hostilities in the summer of 1945. The increase in traffic from the end of 1946 to the present has averaged slightly more than 8 percent compounded annually. From these data it is apparent that the general pattern of traffic growth is being maintained but with some slackening in the rate beginning in 1952.

The lower portion of figure 2, showing the relation of travel by 12-month periods in each of the main geographical regions of the United States to that in the calendar year 1941, shows clearly how much travel was shifted westward during the war period, 1942 to 1946. The spread between the curves for the three regions remained fairly constant during 1947 and 1948, was reduced slightly during 1949 and 1950, and was sharply reduced in the first three months of 1952. In the latter portion of 1952, and the portion of 1953 shown, travel in the Western States seems to be resuming its position in relation to that in the Central and Eastern States.

1952 Summer Loadometer Survey

The check survey in the summer of 1952 was conducted in all respects like those of 1942 to 1951, inclusive. The manner of collecting and analyzing the data in conjunction with that obtained in previous years has been completely described previously.[2] The 1952 survey period, number of stations operated, number of vehicles counted, and the number of trucks and truck combinations weighed are shown for each State in table 1.

Traffic Continues to Increase

Figure 3 shows in chart form the vehicle-mileage of travel on all rural roads, by vehicle types, for each year from 1936 to 1952 inclusive. It is apparent that the drastic restrictions on travel during the war period, 1942–45, caused but a temporary dip in traffic growth and that the 1952 vehicle-mileage was higher than would have been estimated by any rational projection of the prewar trend. A straight line from the top of the bar for 1936 to the top of the bar for 1952 passes through the top of the bar for 1937 and for 1941, but lies above the tops of the bars for all other years. This line indicates an average (simple) annual increase during the 16 years of more than 7.8 percent of the 1936 traffic. During the period 1946 to 1952, inclusive, the average rate of increase was higher than in the earlier period 1936 to 1941. From 1946 to 1952 total travel increased an average amount of more than 10.4 percent of the 1946 figure. The increase in 1952 over 1951 was below the average for the period, but it is probably too early to speculate whether this smaller rate of increase indicates a leveling-off in the trend.

Travel by trucks and truck combinations and by truck combinations alone increased in a manner very similar to that observed for all vehicles. In both cases the straight line, representing the average rate of increase from 1936 to 1952, generally lies above the tops of the bars

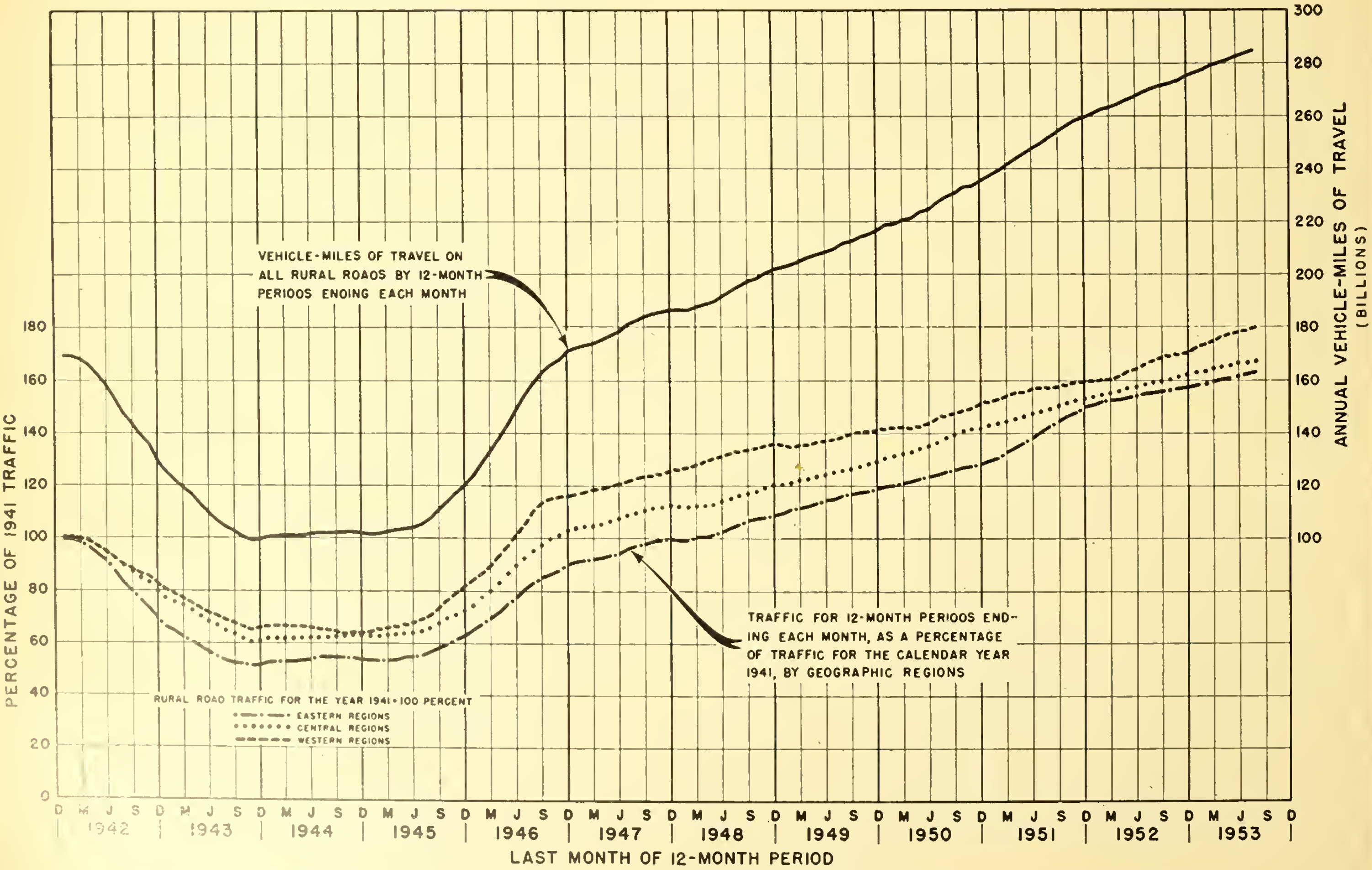

Figure 2.—Travel on all rural roads by 12-month periods ending each month, in vehicle-miles and as a percentage of traffic in the calendar year 1941.

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[2] See previous annual articles on traffic in PUBLIC ROADS: vol. 27, No. 6; vol. 26, Nos. 5 and 11; vol. 25, Nos. 3, 7, and 12; vol. 24, No. 10; and vol. 23, No. 9.

for intervening years, thus showing the accelerated upward trend in travel by these heavier vehicles. This trend is emphasized by data given in other portions of this report.

Travel Increases

The State system of highways in most States is composed of the main rural roads, or those on a connected system carrying the heaviest traffic. In such States as North Carolina, Pennsylvania, and Virginia, where all or a large part of the rural mileage is under State control, only the primary roads are included in this report as a part of the "main" system of highways of the country. These main roads, comprising nearly 361,000 miles, include 12 percent of the total rural mileage but carry 73 percent of the total rural traffic. Because of the greater importance of these highways, from a traffic standpoint, most of the current traffic data were collected at points on them, and the remainder of this report will be concerned only with information concerning this portion of the road mileage.

The data summarized for 1951 and 1952 in this report include, for the first time, the amount of travel on the toll roads of the country. Although the total vehicle-miles of travel on the five major toll roads in these 2 years amounted to less than 1 percent of the total on all main roads, the mileage of toll roads is increasing at such a rate that the omission of their traffic would soon result in an incomplete picture of total traffic.

Although traffic on all rural roads increased over 6 percent, as mentioned before, travel on the main roads alone increased at a slightly higher rate or almost 7 percent. The ratio of traffic volumes on main rural roads in 1952 to corresponding volumes in the previous year is shown in table 2. This table indicates that travel in 1952 on the main highways was higher in every region than in 1951. However, comparison with a similar table comparing 1951 with 1950 shows that the rate of increase for the United States as a whole and for all areas except the Middle Atlantic and the Mountain regions was less than in the previous year. The rate of increase in travel of local passenger cars was considerably less than in 1951, and the rate of foreign (out-of-State) passenger cars and of truck combinations was only slightly higher. Travel by buses on main rural roads was less in 1952 than in the previous year. The rate of increase of all passenger car travel in 1952 was greater than the 1951 rate in the Middle Atlantic and Mountain regions, remained the same in the East North Central and Pacific regions, and was less in all other regions. The rate of increase of all types of truck combinations was less in the Middle Atlantic, the East North Central, the East South Central, the West South Central, and Pacific regions. Foreign travel decreased in volume from 1951 to 1952 in New England, Middle Atlantic, and West South Central and bus travel decreased in New England, Middle Atlantic, East South Central, West North Central, and Mountain regions.

The increase in travel by passenger cars amounted to 6 percent compared to 8 percent for freight-carrying vehicles, and the increase in travel by single-unit trucks in 1952 was 11 percent while that by truck combinations was 2 percent.

The percentage of travel by vehicle types on main rural roads in 1952 is given in table 3. In this table all single-unit trucks are divided into classification types based on the axle and tire arrangements, while the truck combinations are classified according to the total number of axles of the combination. The classification of vehicles into these types has been used in the last 6 annual surveys and has been found to have numerous advantages over the original "light, medium, and

Table 1.—Survey period, number of stations operated, number of vehicles counted, and number weighed in each State in the special weight surveys, summer of 1952

Region and State	Survey period	Number of stations	Total vehicles counted	Trucks and truck combinations	
				Counted	Weighed
New England:					
Connecticut	July 30–Aug. 18	10	36,121	6,455	1,766
Maine	Summer	9	42,428	7,571	3,401
Massachusetts	July 14–Aug. 13	10	111,089	14,085	4,514
New Hampshire	No survey				
Rhode Island	July 16–Aug. 9	4	10,173	2,160	863
Vermont	July 21–July 30	5	8,739	756	756
Subtotal		38	208,550	31,027	11,300
Middle Atlantic:					
New Jersey	July 14–July 29	10	85,783	13,524	1,676
New York	No survey				
Pennsylvania	July 14–Sept. 3	13	82,012	15,923	2,235
Subtotal		23	167,795	29,447	3,911
South Atlantic:					
Delaware	Aug. 7–Aug. 21	8	27,165	5,499	852
Florida	No survey				
Georgia	Aug. 18–Sept. 30	15	37,816	10,077	4,856
Maryland	June 16–July 18	10	118,818	24,361	4,175
North Carolina	Aug. 8–Dec. 8	9	21,059	3,939	3,100
South Carolina	Sept. 8–Sept. 19	10	20,822	5,255	2,088
Virginia	Aug. 5–Aug. 22	5	14,224	3,873	2,383
West Virginia	Aug. 12–Sept. 3	9	14,841	3,479	1,418
Subtotal		66	254,745	56,483	18,872
Eastern regions, subtotal		127	631,090	116,957	34,083
East North Central:					
Illinois	No survey				
Indiana	Aug. 4–Sept. 6	20	56,330	13,204	5,200
Michigan	June 3–July 9	9	31,113	6,529	2,082
Ohio	July 15–July 31	10	29,791	5,166	1,272
Wisconsin	July 1–Sept. 30	20	117,882	18,150	4,132
Subtotal		59	235,116	43,049	12,686
East South Central:					
Alabama	July 15–Aug. 19	10	30,716	6,709	5,965
Kentucky	July 9–Aug. 13	10	36,307	8,746	4,039
Mississippi	June 3–July 18	15	58,430	15,903	7,070
Tennessee	Sept. 3–Sept. 18	8	10,433	3,229	1,740
Subtotal		43	135,886	34,587	18,814
West North Central:					
Iowa	July 21–Aug. 13	10	29,657	5,836	5,789
Kansas	June 6–July 25	10	12,685	2,893	450
Minnesota	June 16–July 3	14	20,787	3,484	1,121
Missouri	July 28–Aug. 27	22	201,680	38,959	13,031
Nebraska	July 17–Aug. 22	20	27,303	5,441	5,365
North Dakota	July 7–Aug. 29	19	36,072	7,407	2,804
South Dakota	June 1–Aug. 31	18	29,263	4,977	4,112
Subtotal		113	357,447	68,997	32,672
West South Central:					
Arkansas	Aug. 1–Aug. 19	10	20,263	6,311	1,450
Louisiana	Aug. 4–Aug. 15	10	13,290	3,922	1,398
Oklahoma	July 9–Aug. 8	10	35,633	7,231	6,008
Texas	June 1–Aug. 31	20	114,525	23,940	6,284
Subtotal		50	183,711	41,404	15,140
Central regions, subtotal		265	912,160	188,037	79,312
Mountain:					
Arizona	July 14–July 25	10	11,775	2,554	1,165
Colorado	July 7–Aug. 29	13	24,813	3,766	805
Idaho	Sept. 8–Oct. 7	13	21,465	4,703	2,249
Montana	July 30–Aug. 31	11	30,003	5,356	2,932
Nevada	Aug. 5–Aug. 28	10	8,470	1,245	986
New Mexico	Aug. 4–Aug. 18	10	16,170	3,625	1,508
Utah	July 21–Aug. 8	10	21,596	3,827	1,165
Wyoming	Aug. 8–Aug. 29	10	14,634	2,812	670
Subtotal		87	148,926	27,888	11,480
Pacific:					
California	June 4–Aug. 13	10	[1] 24,441	4,485	1,959
Oregon	Aug. 6–Aug. 21	7	19,816	4,177	1,468
Washington	June 9–Oct. 6	10	67,270	11,168	6,262
Subtotal		27	111,527	19,830	9,689
Western regions, subtotal		114	260,453	47,718	21,169
United States total		506	1,803,703	352,712	134,564

[1] Passenger cars not counted; figure given is an estimate based on data from other reports.

[illegible] grouping, particularly in that it provides more homogeneous groupings and more positive identification of the types.

The data in table 3 indicate that truck and truck combination travel in 1952 was more than 20 percent of the total travel in all but the New England, Middle Atlantic, and Pacific regions. It was between 20 and 25 percent in all remaining regions except the East South Central and West South Central regions where it was over 25 percent.

A comparison with the same table in the 1951 report shows that the proportion of trucks was higher in 1952 than in 1951 in the New England, South Atlantic, West South Central, Mountain, and Pacific regions and lower in the other regions, although slightly higher in the United States as a whole.

Table 3 indicates also that the usage of certain types of freight-carrying vehicles varies in different sections. For instance, the truck-tractor and semitrailer with five or more axles and the truck and trailer with six or more axles are used far more frequently in the Pacific region than in any other area. Truck and trailer combinations are used much less in the East South Central region and in the three eastern regions than in other sections. The percentage of combination-type vehicles, nation-wide, was 6.49 percent, a slight decrease from the 1951 figure of 6.71 percent. Similar percentage figures in recent years were 7.21 percent in 1950, 5.95 in 1949, and 5.84 in 1948.

The average weights of loaded and empty trucks and truck combinations are shown in figure 4 for each year from 1942 to 1952, inclusive, and for a prewar year, generally 1936 or 1937. The weights of single-unit trucks, both loaded and empty, increased each year from the 1936–37 period through 1945, then leveled off around 11,000 pounds for loaded vehicles and slightly less than 6,000 pounds for empty vehicles. At the same time weights of truck combinations, both loaded and empty, have increased each year during the period shown. The increase in average weight of loaded combinations from the 1936–37 period to 1952 was over 63 percent compared to only about 14 percent for single-unit trucks.

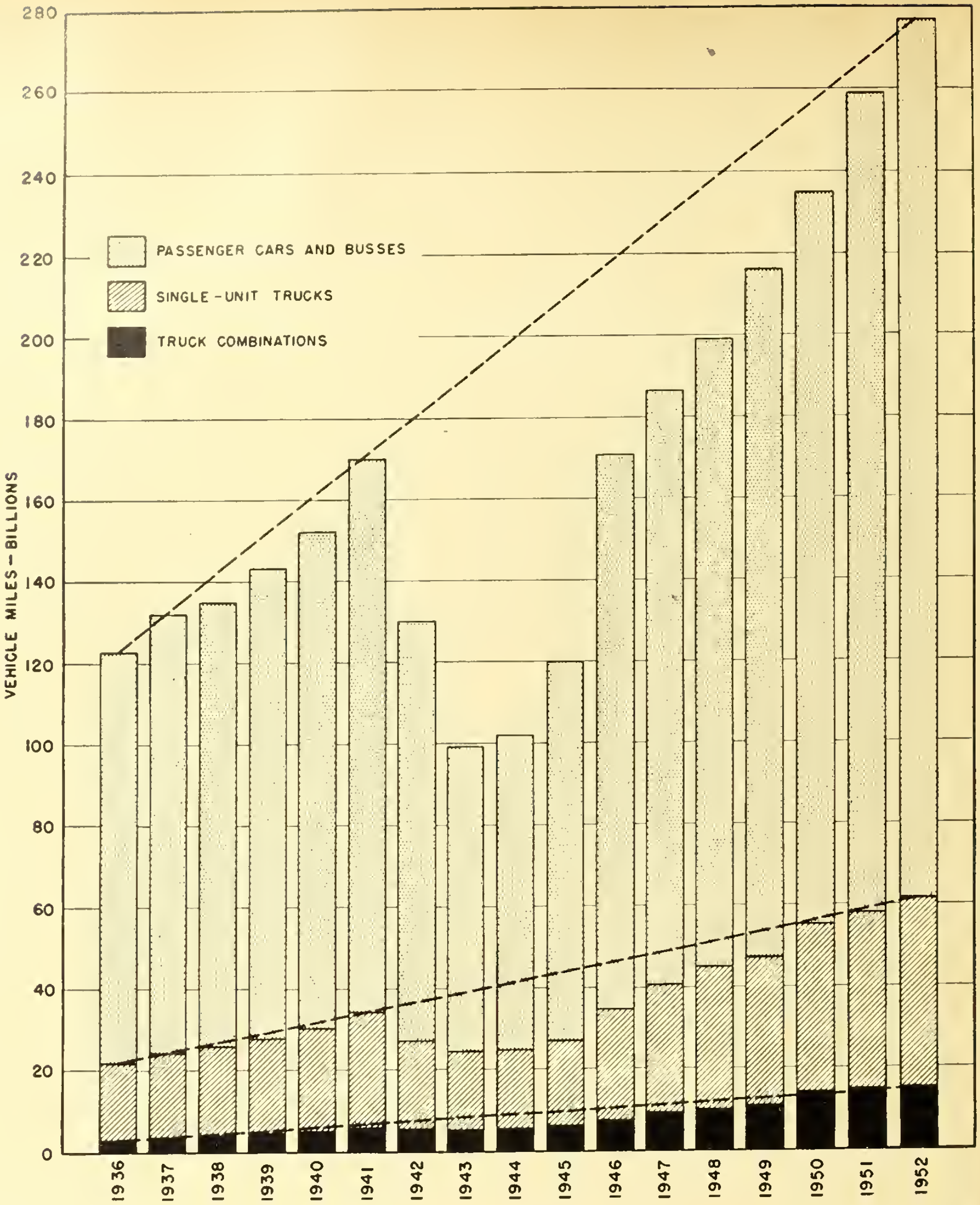


Figure 3.—Travel on all rural roads, 1936–52, by classes of vehicles.

Table 4 gives the average weight of loaded and of empty trucks and truck combinations separately by types in each region and in the United States as a whole. This table brings out clearly the important differences that exist in the weight characteristics of the vehicles in different groups. It will be noted, for example, that for the United States as a whole, the loaded three-axle, single-unit trucks weighed a little more than twice as much as the two-axle, six-tire trucks. The latter, in turn, weighed a little more than twice as much as the two-axle, four-tire trucks. Similar differences existed throughout the various classifications. On the other hand, the regional differences in average weight for each of the vehicle types that are common throughout the country are surprisingly small. The rather low weights of truck

Table 2.—Ratio of 1952 traffic on main rural roads to corresponding traffic in 1951

Vehicle type	Eastern regions [1]				Central regions					Western regions [2]			United States average [1]
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	1.08	1.10	1.01	1.06	1.07	1.18	1.04	1.04	1.07	.98	1.08	1.05	1.06
Foreign	.98	.99	1.14	1.07	1.03	1.01	1.15	.98	1.04	1.27	1.01	1.17	1.07
All passenger cars	1.05	1.08	1.04	1.06	1.06	1.12	1.06	1.03	1.06	1.09	1.07	1.08	1.06
Trucks and truck combinations:													
Single-unit trucks	1.12	1.03	1.20	1.12	1.12	1.02	.95	1.11	1.06	1.33	1.17	1.24	1.11
Truck combinations	1.13	.96	1.12	1.05	.91	1.01	1.19	1.05	1.01	1.28	.94	1.04	1.02
All trucks and combinations	1.12	1.00	1.17	1.10	1.03	1.01	1.01	1.09	1.04	1.32	1.09	1.18	1.08
Buses	.97	.90	1.04	.98	1.00	.95	.95	1.13	1.01	.84	1.01	.95	.99
All Vehicles	1.06	1.06	1.07	1.07	1.05	1.09	1.05	1.04	1.06	1.14	1.07	1.09	1.07

[1] Includes toll road vehicle-mileage for Maine, New Hampshire, New Jersey, and Pennsylvania.
[2] Includes vehicle-mileage for 1952 on the Denver-Boulder toll road, opened Jan. 1952.

Table 3.—Percentage distribution of travel, by vehicle type, on main rural roads in the summer of 1952

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	62.67	66.36	56.38	61.15	56.68	50.06	61.62	60.01	57.67	41.40	69.19	58.64	59.00
Foreign	20.49	13.49	20.47	17.73	22.29	22.21	16.55	13.38	18.72	33.35	10.68	19.29	18.49
All passenger cars	83.16	79.85	76.85	78.88	78.97	72.27	78.17	73.39	76.39	74.75	79.87	77.93	77.49
Single-unit trucks:													
Panel and pickup	4.49	3.74	8.21	5.95	5.73	10.17	6.45	11.09	7.89	10.85	6.12	7.92	7.25
Other 2-axle, 4-tire	.80	1.32	.57	.90	.32	.39	.53	.39	.40	.84	1.25	1.09	.68
Other 2-axle, 6-tire	5.92	7.26	6.31	6.63	6.28	9.62	7.45	7.08	7.25	6.65	4.55	5.35	6.72
3-axle	.36	.47	.76	.59	.42	.48	.24	.15	.32	.73	1.01	.90	.51
All single-unit trucks	11.57	12.79	15.85	14.07	12.75	20.66	14.67	18.71	15.86	19.07	12.93	15.26	15.16
Truck-tractor and semitrailer combinations:													
3-axle	3.63	4.96	3.27	3.99	3.36	4.31	2.71	3.80	3.46	1.34	.78	.99	3.21
4-axle	.49	1.58	2.92	2.07	3.42	1.47	3.12	2.73	2.89	1.12	.88	.97	2.29
5-axle or more	(1)	.02	.03	.02	.21	.03	.50	.09	.22	1.79	2.90	2.48	.54
All truck-tractor and semitrailer combinations	4.12	6.56	6.22	6.08	6.99	5.81	6.33	6.62	6.57	4.25	4.56	4.44	6.04
Truck and trailer combinations:													
4-axle or less	.02	.01	.04	.02	.19	(1)	.18	.33	.19	.31	.27	.29	.15
5-axle	----	.02	----	.01	.34	----	.01	.01	.13	.68	.69	.68	.18
6-axle or more	--	----	----	--	.07	----	----	----	.03	.24	.82	.60	.12
All truck and trailer combinations	.02	.03	.04	.03	.60	(1)	.19	.34	.35	1.23	1.78	1.57	.45
All combinations	4.14	6.59	6.26	6.11	7.59	5.81	6.52	6.96	6.92	5.48	6.34	6.01	6.49
All trucks and truck combinations	15.71	19.38	22.11	20.18	20.34	26.47	21.19	25.67	22.78	24.55	19.27	21.27	21.65
Buses	1.13	.77	1.04	.94	.69	1.26	.64	.94	.83	.70	.86	.80	.86
All Vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	1.00.00	100.00	100.00	100.00

[1] Less than 0.005 percent.

and trailer combinations in the West North Central region indicate a predominance of small homemade trailers of low capacity.

Truck Travel Increases

Figure 5 shows the estimated vehicle-mileage of travel by loaded and empty single-unit trucks and truck combinations on main rural roads for each year, 1936 to 1952, inclusive. This chart demonstrates graphically the steady growth of truck traffic during the prewar years 1936–41, the temporary effect of wartime restrictions in the period 1942–45, and the remarkable increases in truck transportation that have occurred since the end of hostilities in 1945.

Table 5 gives comparisons of the estimated vehicle-mileage of travel by vehicles of different types on all main rural roads in 1936, the earliest year for which comprehensive travel and weight data are available; in 1941, the peak prewar year, 5 years after the beginning of the surveys; in 1946, 10 years after the beginning of the surveys; in 1951, 15 years after the beginning of the sur-

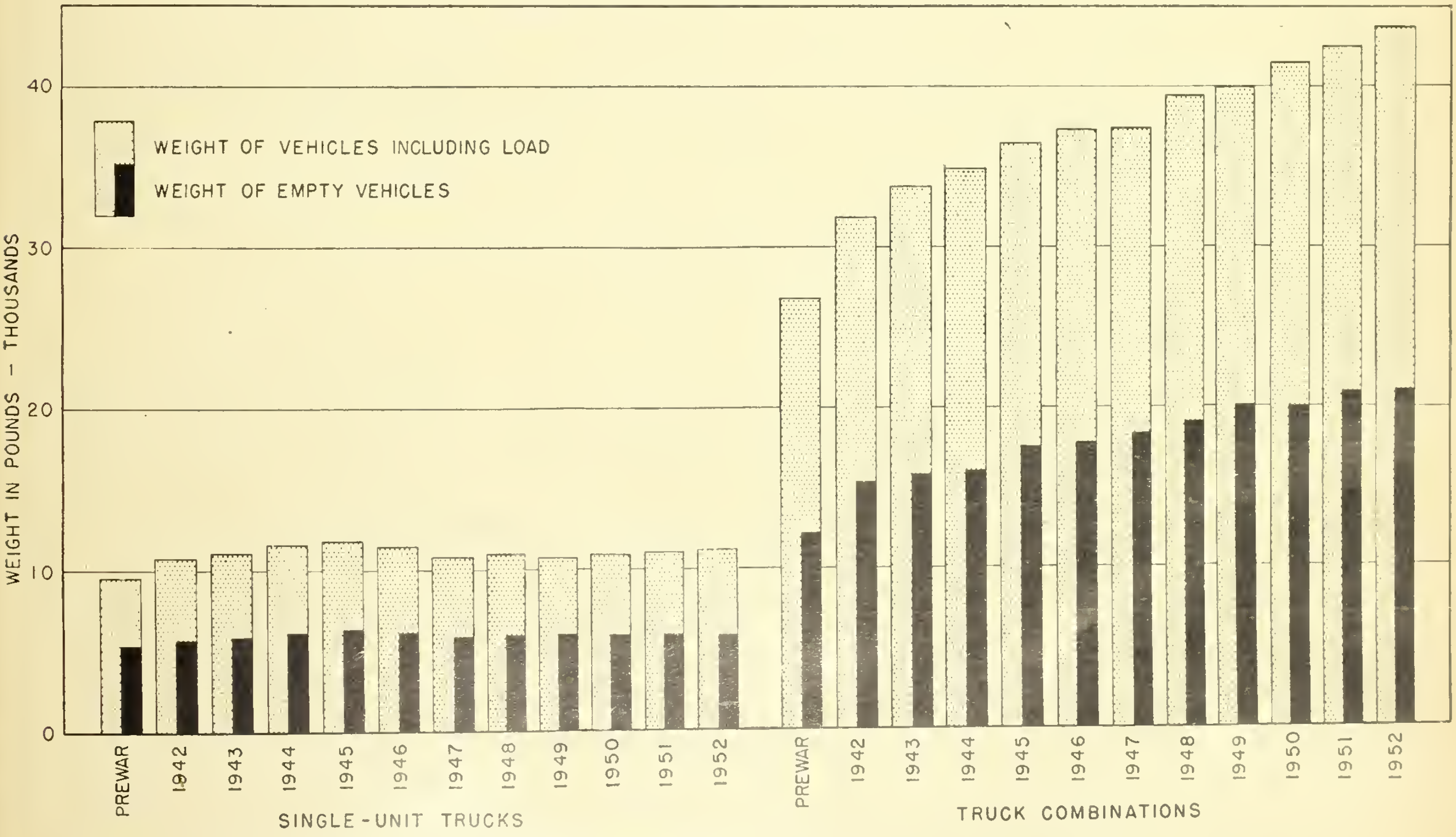


Figure 4.—Average weights of loaded and empty trucks and truck combinations in the summers of 1942–52 and a prewar year.

Table [illegible].—Average weight (in pounds) of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1952

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
AVERAGE WEIGHTS OF LOADED VEHICLES													
Single-unit trucks:													
Panel and pickup	5,198	5,284	5,270	5,263	5,151	5,292	5,271	6,470	5,615	5,160	4,797	4,979	5,397
Other 2-axle, 4-tire	6,856	6,205	7,154	6,727	6,460	6,755	7,513	7,295	7,006	7,087	6,357	6,560	6,757
Other 2-axle, 6-tire	14,554	16,212	13,911	14,940	13,135	14,947	14,195	13,852	13,864	14,655	13,295	13,868	14,204
3-axle	30,492	38,562	30,188	32,604	28,013	28,303	27,575	31,756	28,493	31,294	29,500	29,916	30,378
Average	11,726	14,064	11,646	12,540	10,316	11,990	10,856	10,129	10,643	10,261	11,083	10,730	11,189
Truck combinations:													
Truck-tractor and semitrailer	40,180	41,425	40,315	40,753	40,412	37,613	41,407	41,129	40,489	47,516	57,269	54,025	42,699
Truck and trailer	(1)	(1)		(1)	62,804	(1)	28,785	32,265	51,017	66,767	63,441	64,223	59,647
Average	40,100	41,475	40,315	40,766	41,851	37,614	41,110	40,727	40,949	50,846	58,824	56,342	43,816
Average, all trucks and combinations	20,583	25,487	22,841	23,583	23,376	19,931	21,212	21,265	21,883	21,982	30,401	27,143	23,409
AVERAGE WEIGHTS OF EMPTY VEHICLES													
Single-unit trucks:													
Panel and pickup	4,309	4,271	3,965	4,066	4,122	3,973	4,213	4,639	4,294	4,015	3,963	3,990	4,172
Other 2-axle, 4-tire	5,280	4,555	5,388	4,793	4,786	4,865	5,358	5,412	5,155	5,182	4,984	5,046	4,941
Other 2-axle, 6-tire	8,813	8,741	7,788	8,335	8,090	7,651	8,225	8,281	8,068	8,065	8,032	8,051	8,160
3-axle	15,307	16,170	13,024	14,194	13,144	11,322	14,477	12,362	12,796	16,541	15,953	16,239	14,189
Average	6,590	6,857	5,473	6,063	6,094	5,522	6,103	5,887	5,889	5,644	5,514	5,582	5,903
Truck combinations:													
Truck-tractor and semitrailer	21,987	20,976	20,228	20,724	20,960	16,123	21,277	20,642	20,162	24,598	24,898	24,741	20,660
Truck and trailer	(1)	(1)	(1)	(1)	26,230	(1)	16,948	16,212	19,742	29,941	29,602	29,752	24,682
Average	21,940	21,001	20,196	20,713	21,551	16,125	21,103	20,399	20,135	26,511	26,948	26,733	20,976
Average, all trucks and combinations	9,693	10,574	8,187	9,217	10,829	7,182	9,946	8,607	9,111	8,587	8,981	8,773	9,101

[1] Data omitted because of insufficient sample.

veys; and 1952, the 16th full year of estimates. The ratios of 1952 travel to that of preceding years indicate that increases for trucks and truck combinations were greater than for passenger cars for all years shown, a fact found to be true in most years. The increase of truck-combination travel in 1952 over that in 1951 was less than the increase of single-unit truck travel in the same period, but over the longer periods the increase of travel by truck combinations far exceeds that by single-unit trucks. In the 17 years from 1936 to 1952, passenger car and bus travel combined increased 118 percent, travel by all trucks and combinations increased 185 percent, while travel by truck combinations (considered separately) increased 378 percent, being almost 5 times as great in 1952 as in 1936.

Volume of Highway Freight

Figure 6 gives a comparison of the average load carried by loaded single-unit trucks and truck combinations, separately and combined, in the 17 years that the planning surveys have been operating. The general trend of load weights was upward throughout the period. From 1945 to 1951 the slight decline in the weights of loads carried by single-unit trucks was more than offset by the increased use of combinations and heavier weights of loads carried by vehicles of this type. In 1952, however, the slightly smaller proportion of combinations resulted in a small decline in the average weight of carried load for all trucks and combinations.

Figure 7 shows for each year from 1936 through 1952, the ton-mileage of freight carried by trucks and truck combinations on main rural roads. The chart demonstrates clearly that truck combinations are now transporting a much larger proportion of the total amount of highway freight than in

Table 5.—Comparison of estimated vehicle-miles of travel on main rural roads in 1936, 1941, 1946, 1951, and 1952

Year	All vehicles, vehicle-miles	Passenger cars and buses [1]		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*		*Millions*
1936	88,412	82.6	73,005	17.4	15,407	82.1	12,650	17.9	2,757
1941	122,505	80.3	98,320	19.7	24,185	78.8	19,057	21.2	5,128
1941: 1936 ratio	*1.39*	*.97*	*1.35*	*1.13*	*1.57*	*.96*	*1.51*	*1.18*	*1.86*
1946	124,149	80.4	99,803	19.6	24,346	73.3	17,838	26.7	6,508
1946: 1941 ratio	*1.01*	*1.00*	*1.02*	*.99*	*1.01*	*.93*	*.94*	*1.26*	*1.27*
1946: 1936 ratio	*1.40*	*.97*	*1.37*	*1.13*	*1.58*	*.89*	*1.41*	*1.49*	*2.36*
1951 [2]	190,549	78.6	149,811	21.4	40,738	68.4	27,851	31.6	12,887
1952	203,101	78.4	159,132	21.6	43,969	70.0	30,782	30.0	13,187
1952: 1951 ratio	*1.07*	*1.00*	*1.06*	*1.01*	*1.08*	*1.02*	*1.11*	*.95*	*1.02*
1952: 1941 ratio	*1.66*	*.98*	*1.62*	*1.10*	*1.82*	*.89*	*1.62*	*1.42*	*2.57*
1952: 1936 ratio	*2.30*	*.95*	*2.18*	*1.24*	*2.85*	*.85*	*2.43*	*1.68*	*4.78*

[1] Percentages of total 1952 travel by passenger cars and by buses are reported separately in table 3.
[2] Includes vehicle-miles of travel on toll roads not previously reported.

Table 6.—Comparison of estimated percentage of trucks loaded, average carried load, and ton-miles carried on main rural roads in 1936, 1941, 1946, 1951, and 1952

Year	All truck and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1936	62.8	2.90	28,005	60.7	1.86	14,258	72.2	6.90	13,747
1941	66.7	3.64	58,737	65.4	2.29	28,487	71.6	8.23	30,250
1941: 1936 ratio	*1.06*	*1.26*	*2.10*	*1.08*	*1.23*	*2.00*	*.99*	*1.19*	*2.20*
1946	51.7	4.84	60,892	46.4	2.31	19,101	66.2	9.70	41,791
1946: 1941 ratio	*.78*	*1.33*	*1.04*	*.71*	*1.01*	*.67*	*.92*	*1.13*	*1.38*
1946: 1936 ratio	*.82*	*1.67*	*2.17*	*.76*	*1.24*	*1.34*	*.92*	*1.41*	*3.04*
1951 [1]	55.2	5.68	127,605	48.8	2.31	31,440	68.9	10.83	96,165
1952	54.0	5.56	132,009	48.3	2.34	34,735	67.5	10.93	97,274
1952: 1951 ratio	*.98*	*.98*	*1.03*	*.99*	*1.01*	*1.10*	*.98*	*1.01*	*1.01*
1952: 1941 ratio	*.81*	*1.53*	*2.25*	*.74*	*1.02*	*1.22*	*.94*	*1.33*	*3.22*
1952: 1936 ratio	*.86*	*1.92*	*4.71*	*.80*	*1.26*	*2.44*	*.93*	*1.58*	*7.08*

[1] Includes ton-miles carried on toll roads not previously reported.

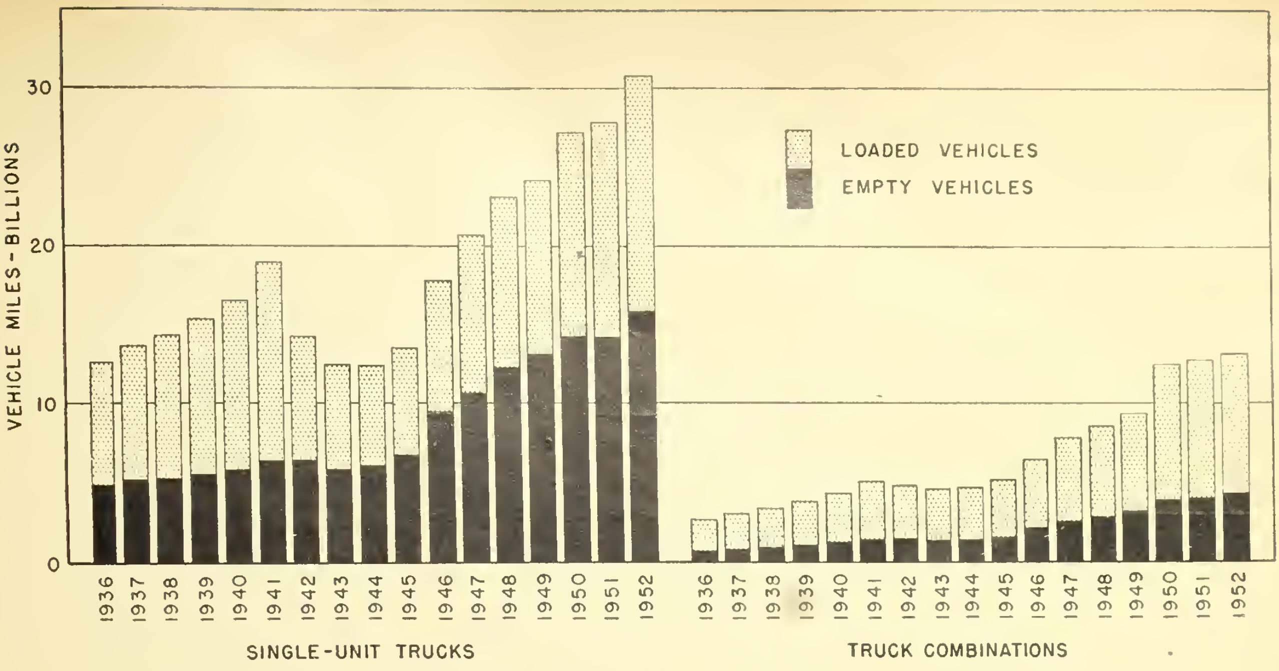


Figure 5.—Travel on main rural roads, 1936–52, by loaded and by empty trucks and truck combinations.

the earlier years. In 1936 the truck combinations hauled slightly less ton-mileage than the single-unit trucks, while in 1952 they hauled almost 3 times the amount transported by the larger number of lighter vehicles. The rapid rate of annual increase in total freight carried, which took place in 1946 and 1947, was reduced somewhat in 1948 and 1949 to a rate more nearly comparable with that of prewar years. In 1950, however, there was a startling increase in freight ton-mileage somewhat similar to the rise that occurred in 1941. In 1951 and 1952, the rate of increase returned to one closely comparable with the prewar trend.

In table 6 are shown comparisons for 1952 with corresponding items for other years as in table 5, of the percentage of vehicles carrying loads, the average carried load, and the ton-mileage carried for single-unit trucks and for truck combinations, separately and combined. The trend from 1936 to 1952 of average weight carried, shown graphically in figure 6, and that of the ton-mileage transported during the same period, shown in figure 7, have already been discussed.

The percentage of truck and truck combinations carrying loads decreased slightly from 1951 to 1952 in the United States as a whole, and in all regions except the New England, East North Central, West North Central, and Mountain regions where slight increases were found. In the country as a whole, the percentage loaded decreased from 55.2 percent in 1951 to 54.0 percent in 1952. Both for single-unit trucks and for truck combinations, the percentage loaded was lower in 1952 than in 1951, but higher than in 1946. The loaded proportion continued to be considerably less for the single-unit vehicles and slightly less for the truck combinations than in the prewar surveys.

Table 7 gives a detailed comparison of the percentage of vehicle-miles of travel, percentage of vehicles loaded, average carried load, and percentage of total ton-miles of freight carried by the various types of trucks and truck combinations traveling on main rural roads in 1951 and 1952. Interesting comparisons can be made from this table showing the relative importance from a freight-carrying standpoint of different portions of the traffic stream. In 1952, for instance, while panel and pickup trucks traveled over 33 percent

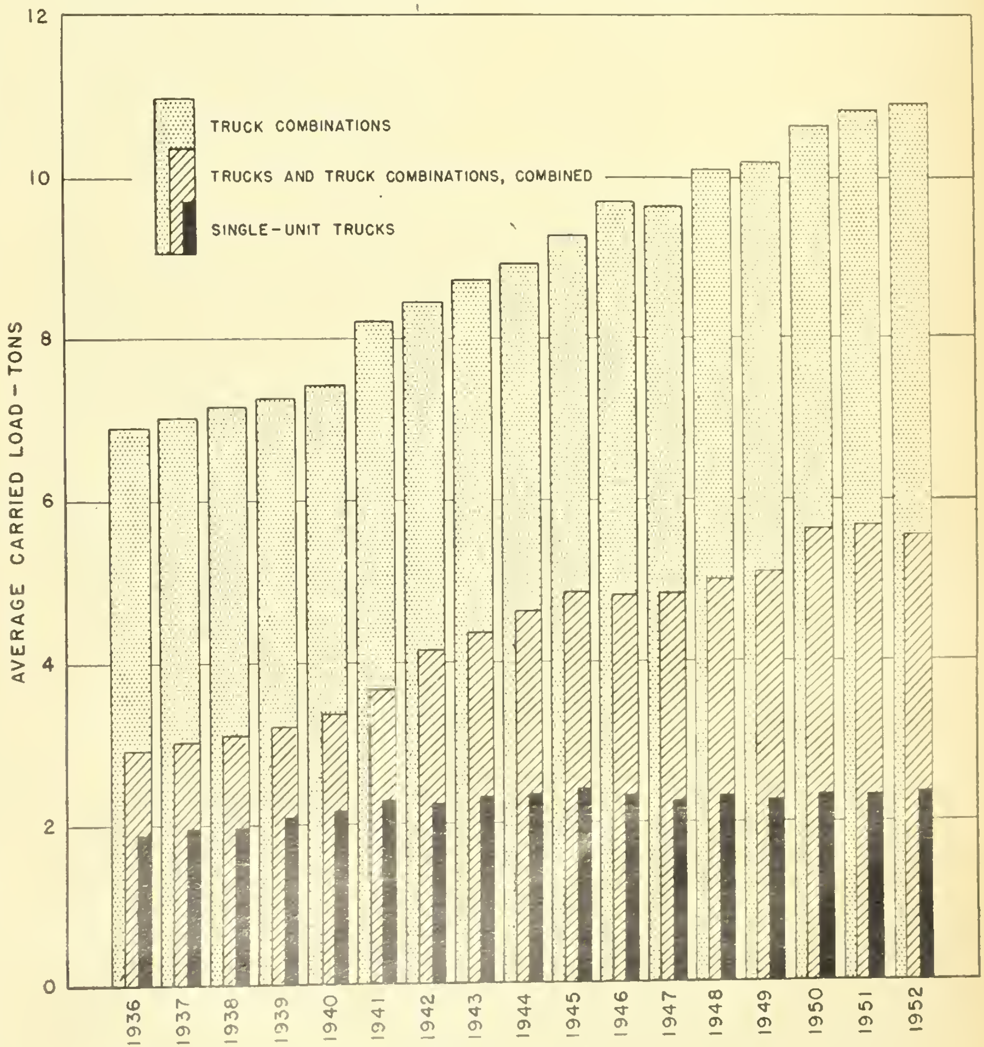


Figure 6.—Average load carried by trucks and truck combinations on main rural roads, 1936–52.

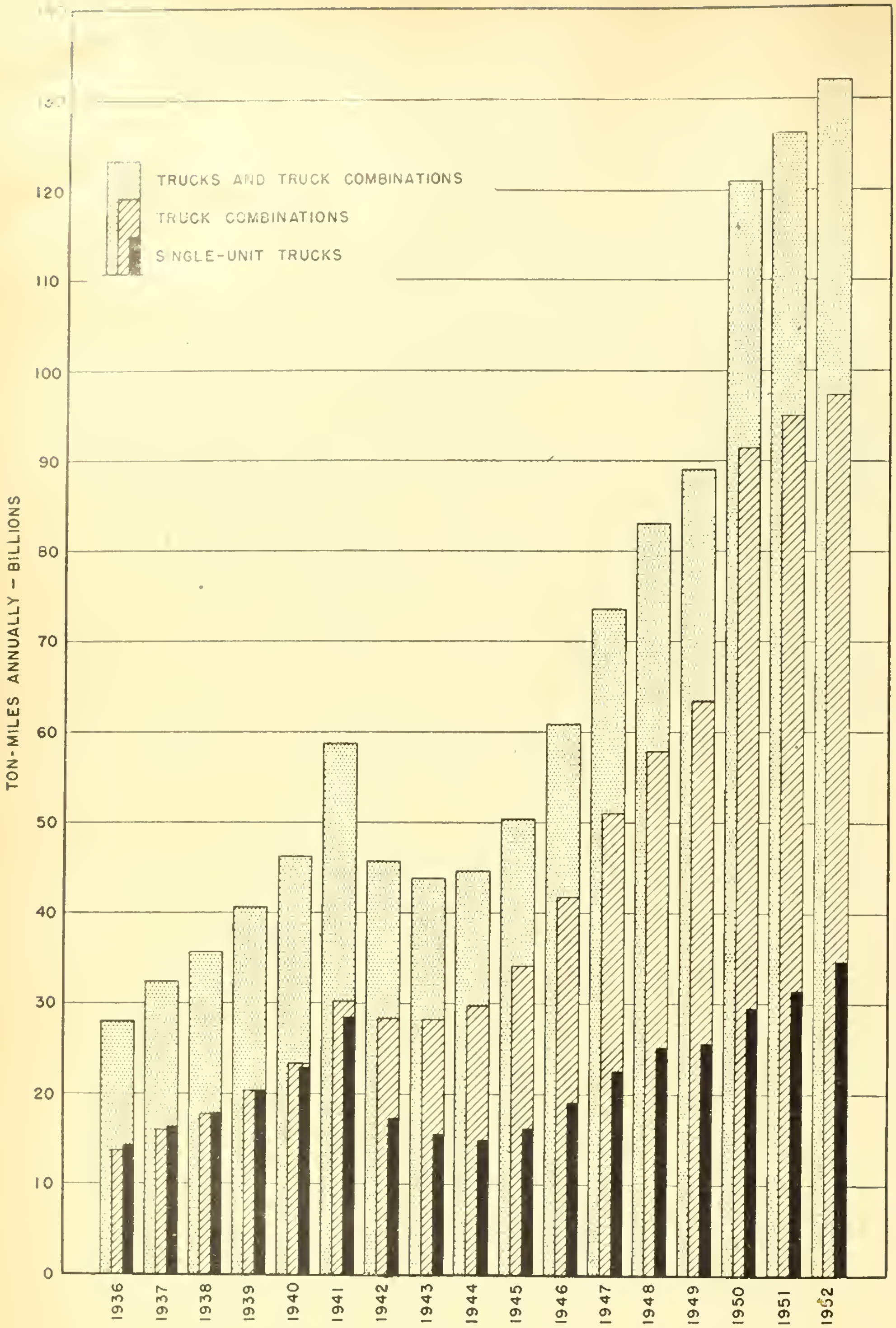


Figure 7.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936–52.

of the vehicle-mileage, they accounted for less than 3 percent of the ton-mileage. The truck-tractors and semitrailers, on the other hand, traveled about 28 percent of the vehicle-mileage but carried almost 67 percent of the ton-mileage.

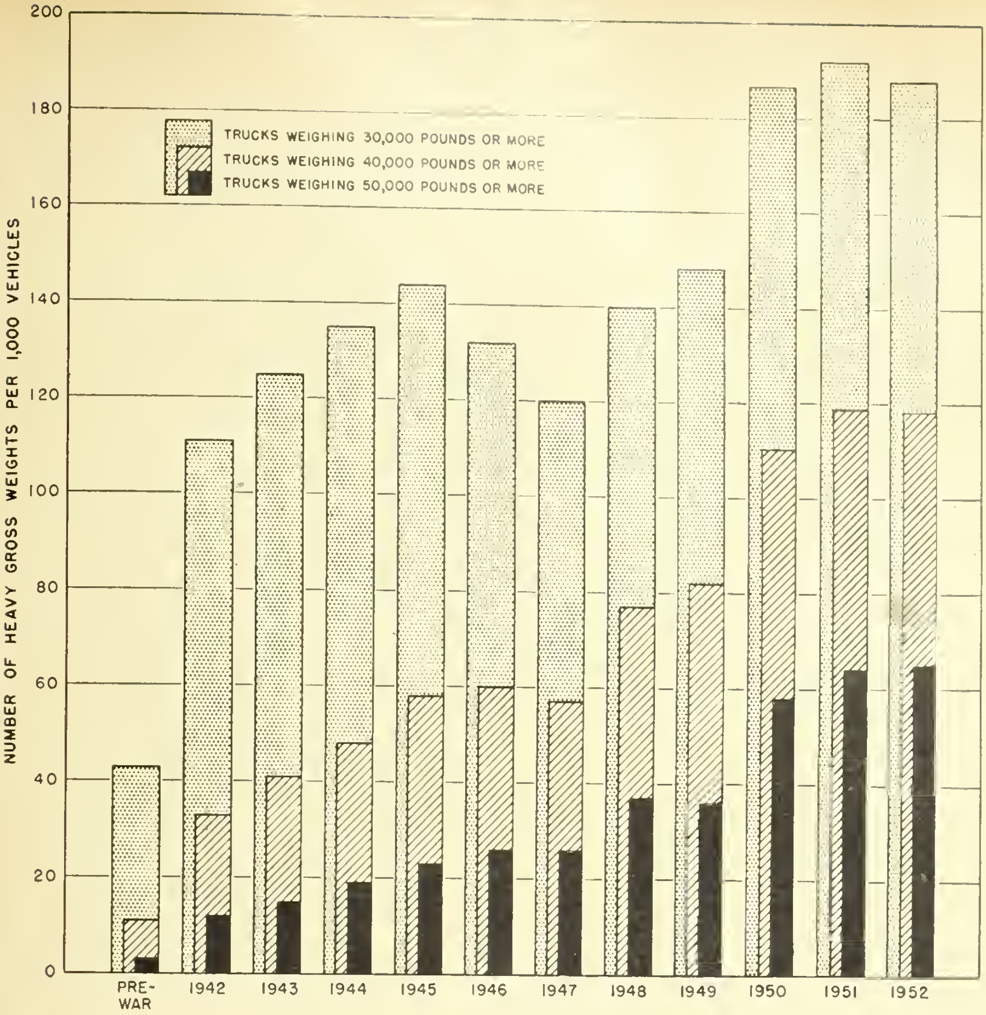
From the column in table 7, showing percentage loaded by types, it can be observed that the percentage of vehicles carrying loads tends to increase directly as the size of the vehicle type, extending from light panel and pickup trucks that are loaded 38 percent of the time to the heavy combinations that are loaded about 67 percent of the time.

Weights Level Off

Figure 8 shows by years, from the prewar years (generally 1936 or 1937) to 1952, for the United States as a whole, the frequency of gross weights of 30,000 pounds or more, of 40,000 pounds or more, and of 50,000 pounds or more. In 1952 it was found that 65 vehicles in each 1,000 loaded and empty trucks and truck combinations weighed 50,000 pounds or more, while in the previous year, 64 vehicles in each 1,000 weighed this amount. On the other hand, it was found in 1952 that 118 vehicles in each 1,000 weighed 40,000 pounds or more, while in 1951, 119 vehicles in each 1,000 weighed that amount. In the 30,000 pound or more class there was a greater reduction. It was found in 1952 that 188 vehicles in each 1,000 weighed 30,000 pounds or more, while in the previous year this frequency was 192 such vehicles. This decline or leveling off of the frequency of heavy gross weights may or may not be significant. It will be seen from the chart that the general upward trend of gross-weight frequencies has been halted or reversed temporarily several times during the period covered, notably in 1946 and 1947. The current frequencies, however, are much larger than those found in 1946, the first postwar year, and an astonishing amount larger than in 1936 or 1937, the first years of the planning surveys. Frequencies of vehicles weighing 30,000 pounds or more in 1952 were 1.4 times as great as in 1946, and over 4 times as great as in the prewar years; those weighing 40,000 pounds or more in 1952 were twice as great as in 1946, and about 11 times as great as in the prewar years; while vehicles weighing 50,000 pounds or more in 1952 were 2.5 times as great as in 1946, and almost 22 times as great as in the prewar years.

The 1952 gross-weight frequency data by vehicle type and region are presented in table 8. No panels, pickups, or other two-axle, four-tire, single-unit trucks were found in the survey weighing as much as 30,000 pounds, so there is no entry for these vehicles in the table though they are included in the total number of vehicles weighed in computing the frequencies for all trucks and combinations. Heavy gross weights are more frequent in the Pacific region than in other parts of the country. In this region 191 of each 1,000 trucks and truck combinations on the main rural highways in 1952, empties included, weighed 50,000 pounds or more, and 280 of each 1,000 weighed 30,000 pounds or more. In the Mountain region 84 of each 1,000 weighed 50,000 pounds or more, while 165 of each 1,000 weighed 30,000 pounds or more. In the East North Central region 78 of each 1,000, a frequency almost as large as that found in the Mountain region, weighed 50,000 pounds or more, while 227 of each 1,000, a frequency almost as large as that found in the Pacific region, weighed 30,000 pounds or more. The lowest frequency of heavy gross loads was found in the East South Central region where only 9 of each 1,000 weighed 50,000 pounds or more, and only 122 of each 1,000 weighed 30,000 pounds or more.

As was pointed out in the discussion of figure 8, the frequencies of heavy gross loads decreased or leveled off in 1952 in the Nation as a whole. This leveling off is not limited to any certain area, but is distributed throughout the entire country. Comparing the frequencies of gross weights in 1952 with those in the previous year, fairly large decreases were found in the frequencies of gross weights of 30,000 pounds or more in the heavily traveled Middle Atlantic and East North Central regions and a slight decrease in the important Pacific region, and with only moderate increases in the other regions. Likewise, decreases were found in the frequencies of gross weights of 50,000 pounds or more in both the Middle Atlantic and East North Central regions and with moderate increases in other regions.



(ABOVE) *Figure 8.—Number of heavy gross weights per 1,000 trucks and truck combinations (empties included) in the summers of 1942–52 and a prewar year.*

(RIGHT) *Figure 9.—Number of heavy axle loads per 1,000 trucks and truck combinations (empties included) in the summers of 1942–52 and a prewar year.*

Frequency of Heavy Axle Loads

Figure 9 shows the frequency of axle loads of 18,000 pounds or more, 20,000 pounds or more, and of 22,000 pounds or more for the prewar years (1936–37) and by years from 1942 to 1952, inclusive. The frequency of these heavy loads increased year by year from the prewar period through 1948. Since 1948, however, the trend apparently has been reversed, for with the exception of 1950, the data seem to indicate a definite downward trend. Such a trend is highly encouraging and may indicate the results of better enforcement of legal limits and of attention given, generally, to better load distribution.

Table 9 gives data concerning the number of heavy axle loads per 1,000 loaded and empty trucks and truck combinations of various types on the main rural roads by regions in 1952. Since no panel, pickup or other two-axle, four-tire single-unit truck was found with axles weighing 18,000 pounds or more, there is no entry for these in the table though they are included in figuring the frequencies for all trucks and truck combinations.

Though the greatest frequency of heavy gross weights in 1952 was in the Pacific region, as shown in table 8, this region had a considerably below-average frequency of heavy axle loads. At the same time, by far the greatest frequency of heavy axle loads was in the Middle Atlantic region and the next greatest was in New England where moderately low frequencies of heavy gross loads were found. In these two eastern regions, the relatively high frequency is attributable mainly to the higher legal axle-weight limits in effect in some of the States, and the large number of two-axle truck-tractors pulling one-axle or two-axle semitrailers. The relative infrequency of heavy axles in the Pacific region, in the presence of a large proportion of heavy gross loads, indicates a better general distribution of the loads over a larger number of axles.

Although the frequency of heavy gross loads has increased considerably in all regions since 1949 and has only showed a tendency to level off in some regions in 1952, as stated in connection with the discussion of table 8, the trend in frequency of heavy axle loads is following a different pattern. This is demonstrated by comparing the frequency of heavy axle loads in 1952 with those in 1951 and 1950 and, in general, back to 1948. As shown in table 9 for each region, with the exception of the New England and Pacific regions, the trend in frequency of heavy axle loads is downward, whereas table 8 shows that the frequency of gross loads is increasing somewhat in most regions and leveling off only in a few.

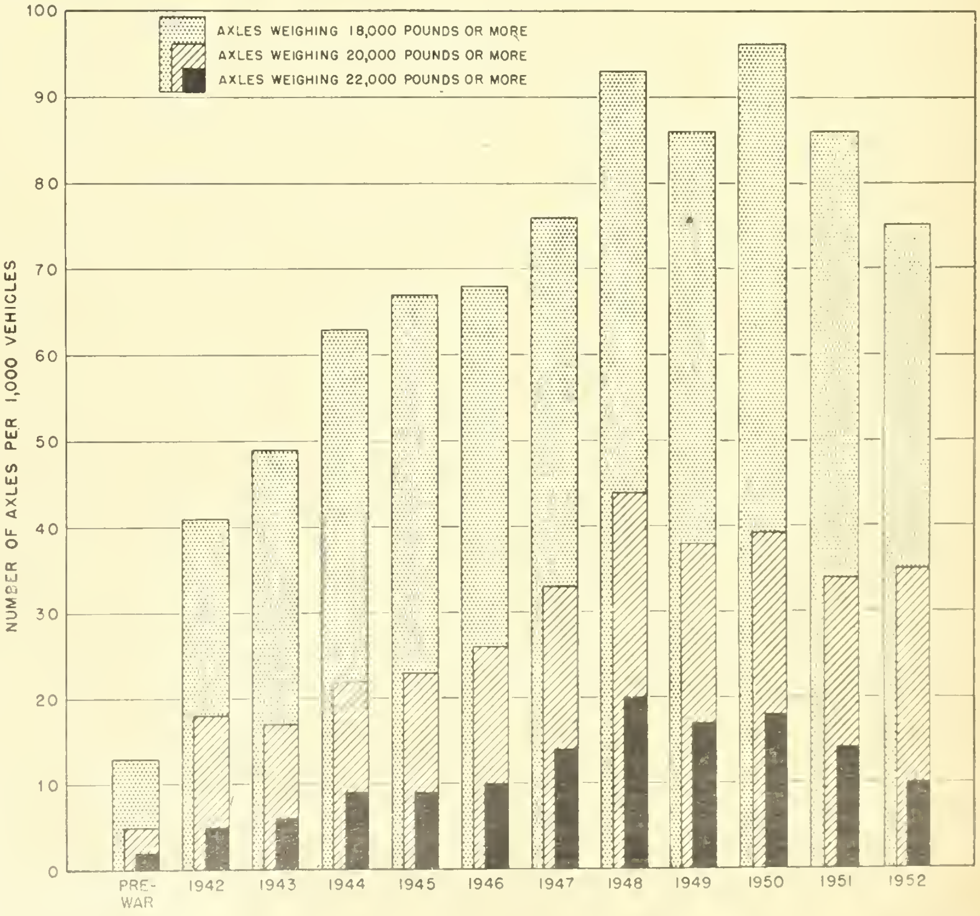


Table .—Percentage of vehicle-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried by various types of trucks and truck combinations on main rural roads in 1952 compared to that in corresponding months in 1951 [1]

Vehicle type	Percentage of vehicle-miles of travel		Percentage loaded		Average carried load		Percentage of ton-miles carried	
	1952	1951	1952	1951	1952	1951	1952	1951
					Tons	*Tons*		
Single-unit trucks:								
Panel and pickup	33.47	32.66	38.1	39.0	0.68	0.70	2.89	2.83
Other 2-axle, 4-tire	3.16	2.89	50.3	54.6	.90	.91	.48	.46
Other 2-axle, 6-tire	31.01	30.85	58.3	58.0	3.20	3.23	19.26	18.42
3-axle	2.36	2.00	58.2	60.9	8.03	7.53	3.68	2.93
All single-unit trucks	70.00	68.40	48.3	48.8	2.34	2.31	26.31	24.64
Truck combinations:								
Truck-tractor and semitrailer	27.89	29.41	67.8	69.2	10.61	10.48	66.84	68.14
Truck and trailer	2.11	2.19	63.5	65.0	15.42	15.48	6.85	7.22
All truck combinations	30.00	31.60	67.5	68.9	10.93	10.83	73.69	75.36
All trucks and combinations	100.00	100.00	54.0	55.2	5.56	5.68	100.00	100.00

[1] Data for 1951 include estimates for toll roads not previously reported.

Loads Above Legal Limits

Table 10 shows the number of trucks and truck combinations of each type, per 1,000 such vehicles counted, empties included, that exceeded the legal axle, axle-group, or gross-weight limits in effect in the individual States in the summer of 1952, and the number per 1,000 that exceeded these limits by various percentages. Comparative figures are given at the bottom of the table for the Nation as a whole for 1950 and 1951. The table shows that, on the whole, the frequency with which vehicles violated some one of the State weight restrictions was approximately the same as in the previous year, much less than in 1950, and furthermore, that the amount of overload generally was less in 1952 than in the previous years.

Loads in excess of State law in 1952 were most frequent in the Middle Atlantic States, although even in this region, when compared to 1951, a reduction in frequency of overloaded vehicles was found. The frequencies were reduced from 74 trucks and combinations overloaded in 1951 for each 1,000 counted to a frequency of 61 such vehicles in 1952. A slight decrease in the frequencies of overloaded vehicles also was found in the East North Central and in the West North Central regions, and the frequency in the West South Central region remained the same as in 1951. Slight increases in the frequency of overloaded vehicles were found in all other areas. The East South Central region, which had the second lowest frequency of overloaded vehicles in 1951, had the lowest such frequency in 1952 with only 44 trucks and truck combinations overloaded for each 1,000 counted in that area. After the Middle Atlantic region where, of all loaded and empty trucks and truck combinations weighed in 1952, 61 of each 1,000 exceeded one or more of the State weight limits, there followed the East North Central region with the second highest rate of overloads (57), and in descending order of rates of violation were the West South Central (54), the Mountain (54), the Pacific (51), the South Atlantic (49), the West North Central (48), New England (46), and the East South Central (44).

No panel or pickup or other 2-axle, 4-tire truck was weighed that exceeded any of the State weight regulations, and these classifications are

Table 8.—Heavy gross weights per 1,000 loaded and empty trucks and truck combinations on main rural roads, summer of 1952

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 30,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	16	16	1	9	------	1	------	3	1	2	------	1	4
3-axle	326	367	346	351	239	270	294	478	283	331	373	360	333
Average	18	22	17	19	8	7	5	5	6	13	29	22	13
Truck combinations:													
Truck-tractor and semitrailer	549	566	563	563	589	533	588	550	571	679	786	747	591
Truck and trailer	(1)	(1)	------	(1)	681	400	231	208	511	737	809	788	666
Average	547	565	559	560	596	533	577	534	568	692	793	758	596
Average, all trucks and combinations	158	207	171	183	227	122	181	148	177	165	280	230	188
Comparative average, 1951	153	234	164	191	255	109	158	143	178	164	289	240	192
Comparative average, 1950	137	221	177	189	251	102	142	146	170	160	289	233	187
NUMBER PER 1,000 WEIGHING 40,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	2	2	------	1	------	------	------	------	------	(2)	------	(2)	(2)
3-axle	124	152	58	92	114	25	27	111	79	59	31	40	72
Average	5	7	3	4	4	1	(2)	1	2	2	2	2	2
Truck combinations:													
Truck-tractor and semitrailer	357	341	363	353	353	221	377	353	341	469	664	594	377
Truck and trailer	(1)	(1)	------	(1)	551	200	168	37	379	527	620	592	500
Average	356	341	361	352	369	221	371	338	343	482	652	593	386
Average, all trucks and combinations	97	120	104	110	140	49	115	92	105	110	216	169	118
Comparative average, 1951	95	152	98	120	154	41	98	85	103	103	216	172	119
Comparative average, 1950	78	135	95	109	140	45	82	79	95	106	214	167	110
NUMBER PER 1,000 WEIGHING 50,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	------	------	------	------	------	------	------	------	------	------	------	------	------
3-axle	21	38	1	14	21	12	------	23	16	9	3	5	12
Average	1	1	(2)	1	1	(2)	------	(2)	(2)	(2)	(2)	(2)	(2)
Truck combinations:													
Truck-tractor and semitrailer	141	108	154	128	184	41	209	180	170	344	579	494	197
Truck and trailer	------	(1)	------	(1)	486	------	152	31	334	487	583	555	460
Average	140	109	153	128	208	41	207	173	178	376	580	510	215
Average, all trucks and combinations	38	38	43	39	78	9	64	47	54	84	191	144	65
Comparative average, 1951	31	72	36	50	87	6	54	34	51	78	178	138	64
Comparative average, 1950	24	63	28	41	78	7	44	34	47	76	176	133	58

[1] Data omitted because of insufficient sample.

[2] Less than 5 per 10,000.

omitted from tables 10–12 although the number of such vehicles counted is included in the calculations.

Recommended Weight Limits

Uniform regulations concerning maximum allowable gross weights, axle weights, and axle-group weights have been adopted as a policy by the American Association of State Highway Officials and recommended to the State governments for adoption.[3] This policy recommends that no axle shall carry a load in excess of 18,000 pounds, and no group of axles shall carry a load in excess of amounts specified in a table of permissible weights based on the distance between the extremes of any group of axles.

As might be expected, many vehicles were loaded in such a manner that they exceeded more than one recommended weight limit, and some vehicles had more than one axle loaded in excess of the recommended limit. By counting each vehicle only once, regardless of the number of ways in which it exceeded any of the A.A.S.H.O. recommended limits, table 11 was prepared to show the number of vehicles per 1,000 of each type, both loaded and empty, that exceeded the limits by various percentages. Those vehicles which exceeded more than one provision of the recommended restriction were tabulated only in the column showing the highest percentage excess of any item.

In the United States as a whole, the number of vehicles out of every 1,000 that exceeded the recommendations increased slightly, being 73 in 1952 compared to 72 in 1951. A slight reduction in the number exceeding the recommendations was found in the West North Central region. On the other hand, a considerable reduction was found in the important Middle Atlantic and East North Central regions where the number decreased from 127 and 99 in 1951 to 107 and 89 in 1952, respectively. In all other regions, slight increases were found, the largest of which were in the New England and Pacific regions. The large reduction of vehicles exceeding the recommendations in the Middle Atlantic region is particularly gratifying because this is the region of the highest frequency of excessive loads, as measured by the A.A.S.H.O. standards.

Regardless of the reduction in the Middle Atlantic region of the frequency of vehicles out of every 1,000 that exceeded any of the A.A.S.H.O. recommendations in 1952, when compared to 1951, that region led all others in the number of heavy loads. There were 107 vehicles for every 1,000 weighed, including empties, that exceeded one or more of the recommendations. Other regions, ranked in descending order, were New England (103), Pacific (101), East North Central (89), Mountain (69), South Atlantic (65), West South Central (54), West North Central (46), and the East South Central (38).

In the United States as a whole, 73 vehicles out of every 1,000 were overloaded to some degree according to the A.A.S.H.O. standards, and 14 out of every 1,000 exceeded some one of the recommended provisions by 20 percent or more in 1952. Compared to these frequencies, there were in the previous year 72 vehicles out of every 1,000 that were overloaded to some degree, and 16 out of every 1,000 were overloaded by 20 percent or more. Although these data indicate a slight increase in the number of vehicles loaded in excess of the Association's recommendations, there is a gratifying reduction in the number of the more heavily overloaded vehicles.

The frequencies of axle loads of 18,000 pounds or more, 20,000 pounds or more, and 22,000 pounds or more have already been discussed in connection with table 9 and will not be discussed further.

Table 12 shows the number of vehicles of various types per 1,000 vehicles with an axle-group load in excess of the limits recommended by the A.A.S.H.O. and in excess of the limits by various percentages. For the United States as a whole,

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[3] *Policy concerning maximum dimensions, weights, and speeds of motor vehicles to be operated over the highways of the United States*, adopted April 1, 1946, by the American Association of State Highway Officials; published by the Association in 1946.

Table 9.—Heavy axle loads per 1,000 loaded and empty trucks and truck combinations on main rural roads, summer of 1952

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 18,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	45	53	28	41	13	33	11	21	18	39	23	30	27
3-axle	190	158	76	111	34	83	10	118	50	84	19	39	70
Average	30	36	15	24	8	17	6	9	9	17	9	13	14
Truck combinations:													
Truck-tractor and semitrailer	499	395	242	330	176	147	96	207	162	158	135	143	216
Truck and trailer	(1)	------	------	(1)	372	------	42	12	244	227	214	218	223
Average	497	393	240	328	192	147	94	198	167	173	157	163	217
Average, all trucks and combinations	153	157	79	116	76	46	33	60	57	52	58	55	75
Comparative average, 1951	151	207	90	144	88	46	53	60	65	55	44	49	86
Comparative average, 1950	137	208	100	147	98	63	45	67	72	83	69	75	96
NUMBER PER 1,000 WEIGHING 20,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	29	28	8	19	1	13	2	8	5	13	4	9	10
3-axle	77	107	12	47	25	16	----	23	19	21	8	12	27
Average	17	20	4	11	1	6	1	3	3	5	2	4	6
Truck combinations:													
Truck-tractor and semitrailer	305	207	79	254	50	27	15	57	41	33	25	28	111
Truck and trailer	(1)	------	------	(1)	38	------	2	------	24	11	40	31	28
Average	304	206	78	252	49	27	14	54	40	28	29	29	105
Average, all trucks and combinations	93	83	25	84	19	11	5	17	14	11	11	11	35
Comparative average, 1951	97	129	35	79	16	11	12	21	16	17	5	10	34
Comparative average, 1950	82	131	38	80	22	19	12	23	19	35	16	24	39
NUMBER PER 1,000 WEIGHING 22,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	19	16	2	10	----	2	(2)	4	1	7	–	3	5
3-axle	28	49	2	19	13	----	----	18	8	4	(2)	2	10
Average	11	11	1	6	(2)	1	(2)	2	1	3	(2)	1	2
Truck combinations:													
Truck-tractor and semitrailer	157	114	23	74	9	7	2	12	8	8	6	6	30
Truck and trailer	(1)	------	------	(1)	15	------	------	------	10	2	7	5	7
Average	156	113	23	73	9	7	2	11	8	6	6	6	28
Average, all trucks and combinations	49	46	7	26	4	2	1	4	3	3	2	3	10
Comparative average, 1951	46	73	10	39	5	3	3	4	4	5	2	3	14
Comparative average, 1950	39	80	13	42	7	5	3	9	6	16	3	9	18

[1] Data omitted because of insufficient sample.

[2] Less than 5 per 10,000.

Table 10.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle, axle-group, or gross-weight legal limits in effect in the States by various percentages (maximum) of overload, summer of 1952

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England						
2-axle, 6-tire	16	13	8	4	2	(1)
3-axle	132	106	73	16	12	2
Average, single-unit trucks	12	10	6	3	1	(1)
Truck-tractor and semitrailer	142	101	73	29	10	(1)
Truck and trailer	----	----	----	----	----	----
Average, truck combinations	141	101	73	29	10	(1)
Average, all trucks and combinations	46	34	24	10	3	(1)
Middle Atlantic:						
2-axle, 6-tire	23	18	12	5	2	1
3-axle	116	92	77	54	38	15
Average, single-unit trucks	17	14	10	5	3	1
Truck-tractor and semitrailer	145	90	56	15	6	(1)
Truck and trailer	187	125	62	----	----	----
Average, truck combinations	145	90	56	15	6	(1)
Average, all trucks and combinations	61	40	26	8	4	1
South Atlantic:						
2-axle, 6-tire	15	6	4	1	----	----
3-axle	70	21	14	1	----	----
Average, single-unit trucks	9	3	2	(1)	----	----
Truck-tractor and semitrailer	150	88	52	19	7	1
Truck and trailer	----	----	----	----	----	----
Average, truck combinations	149	87	52	19	7	1
Average, all trucks and combinations	49	27	16	5	2	(1)
East North Central:						
2-axle, 6-tire	5	1	----	----	----	----
3-axle	49	28	27	13	13	----
Average, single-unit trucks	4	1	1	(1)	(1)	----
Truck-tractor and semitrailer	140	79	36	8	1	(1)
Truck and trailer	223	120	58	13	7	----
Average, truck combinations	147	82	38	8	1	(1)
Average, all trucks and combinations	57	31	15	3	(1)	(1)
East South Central:						
2-axle, 6-tire	28	20	14	3	1	----
3-axle	68	48	31	14	----	----
Average, single-unit trucks	15	10	7	2	(1)	----
Truck-tractor and semitrailer	145	84	46	16	3	1
Truck and trailer	----	----	----	----	----	----
Average, truck combinations	145	84	46	16	3	1
Average, all trucks and combinations	44	26	16	5	1	(1)
West North Central:						
2-axle, 6-tire	7	3	1	(1)	----	----
3-axle	55	32	22	11	----	----
Average, single-unit trucks	4	2	1	(1)	----	----
Truck-tractor and semitrailer	148	93	47	12	3	(1)
Truck and trailer	66	37	9	----	----	----
Average, truck combinations	146	91	46	12	3	(1)
Average, all trucks and combinations	48	29	15	4	1	(1)
West South Central:						
2-axle, 6-tire	17	13	7	5	2	1
3-axle	101	88	59	17	----	----
Average, single-unit trucks	7	6	3	2	1	(1)
Truck-tractor and semitrailer	189	126	77	26	9	1
Truck and trailer	4	----	----	----	----	----
Average, truck combinations	180	120	73	25	9	1
Average, all trucks and combinations	54	37	22	8	3	(1)
Mountain:						
2-axle, 6-tire	31	21	11	5	2	----
3-axle	92	87	48	22	13	----
Average, single-unit trucks	14	11	6	3	1	----
Truck-tractor and semitrailer	189	122	74	22	7	1
Truck and trailer	218	157	83	29	18	----
Average, truck combinations	195	130	76	24	9	1
Average, all trucks and combinations	54	38	22	8	3	(1)
Pacific:						
2-axle, 6-tire	6	4	3	----	----	----
3-axle	24	7	4	----	----	----
Average, single-unit trucks	4	2	1	----	----	----
Truck-tractor and semitrailer	125	80	41	16	8	3
Truck and trailer	201	92	44	11	6	4
Average, truck combinations	146	83	42	15	7	3
Average, all trucks and combinations	51	29	14	5	2	1
United States average:						
2-axle, 6-tire	15	10	6	2	1	(1)
3-axle	67	41	30	13	8	2
Average, single-unit trucks	9	6	4	1	1	(1)
Truck-tractor and semitrailer	151	93	52	16	5	1
Truck and trailer	182	98	48	13	7	2
Average, truck combinations	153	93	52	16	5	1
Average, all trucks and combinations	52	32	18	6	2	(1)
Comparative average, 1951	50	32	19	7	3	(1)
Comparative average, 1950	67	44	27	11	5	1

[1] Less than 5 per 10,000.

Table 11.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the permissible load limits recommended by the A.A.S.H.O. by various percentages (maximum) of overload in the summer of 1952

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 6-tire	47	40	32	23	14	3
3-axle	175	151	126	59	18	----
Average, single-unit trucks	29	25	20	14	8	2
Truck-tractor and semitrailer	312	273	230	143	69	11
Truck and trailer	----	----	----	----	----	----
Average, truck combinations	311	272	229	142	69	11
Average, all trucks and combinations	103	90	75	48	24	4
Middle Atlantic:						
2-axle, 6-tire	45	38	32	18	10	2
3-axle	100	85	77	54	38	23
Average, single-unit trucks	29	25	21	12	7	2
Truck-tractor and semitrailer	257	212	152	88	49	9
Truck and trailer	187	187	125	----	----	----
Average, truck combinations	257	212	152	88	49	9
Average, all trucks and combinations	107	89	66	38	21	4
South Atlantic:						
2-axle, 6-tire	23	13	8	1	(1)	----
3-axle	122	89	44	5	1	----
Average, single-unit trucks	15	9	5	1	(1)	----
Truck-tractor and semitrailer	194	138	89	35	11	2
Truck and trailer	----	----	----	----	----	----
Average, truck combinations	193	137	88	35	11	2
Average, all trucks and combinations	65	45	29	11	3	1
East North Central:						
2-axle, 6-tire	6	1	----	----	----	----
3-axle	41	27	14	13	12	----
Average, single-unit trucks	4	1	(1)	(1)	(1)	----
Truck-tractor and semitrailer	215	132	75	19	7	(1)
Truck and trailer	445	406	323	164	66	12
Average, truck combinations	233	154	95	30	12	1
Average, all trucks and combinations	89	58	35	11	4	(1)
East South Central:						
2-axle, 6-tire	28	20	15	4	(1)	----
3-axle	70	61	35	11	----	----
Average, single-unit trucks	15	11	8	2	(1)	----
Truck-tractor and semitrailer	120	66	34	10	2	(1)
Truck and trailer	----	----	----	----	----	----
Average, truck combinations	120	66	34	10	2	(1)
Average, all trucks and combinations	38	23	14	4	(1)	(1)
West North Central:						
2-axle, 6-tire	8	3	1	(1)	----	----
3-axle	24	21	17	2	----	----
Average, single-unit trucks	4	2	1	(1)	----	----
Truck-tractor and semitrailer	144	82	35	9	2	----
Truck and trailer	56	29	6	----	----	----
Average, truck combinations	141	80	34	9	2	----
Average, all trucks and combinations	46	26	11	3	1	----
West South Central:						
2-axle, 6-tire	17	13	7	5	2	1
3-axle	91	78	54	17	----	----
Average, single-unit trucks	7	6	3	2	1	(1)
Truck-tractor and semitrailer	190	128	79	27	9	1
Truck and trailer	4	4	4	4	----	----
Average, truck combinations	181	122	75	26	9	1
Average, all trucks and combinations	54	37	23	9	3	(1)
Mountain:						
2-axle, 6-tire	31	21	10	5	2	----
3-axle	90	72	43	17	4	----
Average, single-unit trucks	14	10	5	2	1	----
Truck-tractor and semitrailer	251	179	114	34	12	1
Truck and trailer	293	229	132	31	(1)	----
Average, truck combinations	260	190	118	33	9	1
Average, all trucks and combinations	69	50	30	9	3	(1)
Pacific:						
2-axle, 6-tire	6	4	3	----	----	----
3-axle	19	6	4	----	----	----
Average, single-unit trucks	4	2	1	----	----	----
Truck-tractor and semitrailer	261	204	129	40	13	3
Truck and trailer	398	300	190	20	9	4
Average, truck combinations	299	231	146	34	12	3
Average, all trucks and combinations	101	77	49	11	4	1
United States average:						
2-axle, 6-tire	21	15	10	5	2	1
3-axle	74	56	36	15	8	3
Average, single-unit trucks	12	9	6	3	1	1
Truck-tractor and semitrailer	207	146	92	37	16	2
Truck and trailer	330	267	183	54	20	5
Average, truck combinations	216	154	98	38	16	2
Average, all trucks and combinations	73	52	34	14	6	1
Comparative average, 1951	72	52	35	16	7	1
Comparative average, 1950	91	68	46	21	10	3

[1] Less than 5 per 10,000.

the frequency of excessive axle-group loads in 1952 was slightly higher than in 1951, although lower than in 1950. As in table 11, decreases in the frequency of vehicles exceeding the axle-group recommendations were found in the heavily traveled Middle Atlantic and East North Central regions, and small increases were found in all others. As in the previous year, the highest frequency of excessive axle-group loads was found in the Pacific region (91 per 1,000 vehicles), while the regions in descending order of the number of vehicles with excessive axle-group loads were East North Central (61), Mountain (54), West North Central (33), Middle Atlantic (32), South Atlantic (30), New England and West South Central (28 each), and East South Central (6).

It will be noted that a higher proportion of the vehicles have excessive axle-group loads in the

Table 12.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle-group loads recommended by the A.A.S.H.O. by various percentages of overload in the summer of 1952

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 6-tire	2	1	1	—	—	—
3-axle	131	109	88	47	13	—
Average, single-unit trucks	5	4	3	1	(1)	—
Truck-tractor and semitrailer	93	64	43	20	6	1
Truck and trailer	—	—	—	—	—	—
Average, truck combinations	93	64	43	20	6	1
Average, all trucks and combinations	28	20	14	6	2	(1)
Middle Atlantic:						
2-axle, 6-tire	—	—	—	—	—	—
3-axle	100	85	77	54	38	15
Average, single-unit trucks	4	3	3	2	1	1
Truck-tractor and semitrailer	83	58	34	16	7	2
Truck and trailer	187	187	125	—	—	—
Average, truck combinations	83	58	34	16	7	2
Average, all trucks and combinations	32	22	14	7	3	1
South Atlantic:						
2-axle, 6-tire	—	—	—	—	—	—
3-axle	114	83	35	5	1	—
Average, single-unit trucks	5	4	2	(1)	(1)	—
Truck-tractor and semitrailer	96	65	41	14	6	1
Truck and trailer	—	—	—	—	—	—
Average, truck combinations	95	65	41	14	6	1
Average, all trucks and combinations	30	21	13	4	2	(1)
East North Central:						
2-axle, 6-tire	—	—	—	—	—	—
3-axle	38	27	14	13	13	—
Average, single-unit trucks	1	1	(1)	(1)	(1)	—
Truck-tractor and semitrailer	141	84	44	11	4	(1)
Truck and trailer	404	387	321	160	65	12
Average, truck combinations	162	108	66	23	9	1
Average, all trucks and combinations	61	41	25	9	3	(1)
East South Central:						
2-axle, 6-tire	—	—	—	—	—	—
3-axle	54	47	30	11	—	—
Average, single-unit trucks	1	1	1	(1)	—	—
Truck-tractor and semitrailer	23	10	5	1	(1)	(1)
Truck and trailer	—	—	—	—	—	—
Average, truck combinations	23	10	5	1	(1)	(1)
Average, all trucks and combinations	6	3	2	(1)	(1)	(1)
West North Central:						
2-axle, 6-tire	(1)	(1)	—	—	—	—
3-axle	22	21	17	2	—	—
Average, single-unit trucks	(1)	(1)	(1)	(1)	—	—
Truck-tractor and semitrailer	108	62	23	6	1	—
Truck and trailer[1]	42	2	—	—	—	—
Average, truck combinations	106	60	22	6	1	—
Average, all trucks and combinations	33	18	7	2	(1)	—
West South Central:						
2-axle, 6-tire	—	—	—	—	—	—
3-axle	83	70	45	8	—	—
Average, single-unit trucks	1	1	(1)	(1)	—	—
Truck-tractor and semitrailer	104	73	43	17	5	1
Truck and trailer	4	4	4	4	—	—
Average, truck combinations	99	70	41	16	5	1
Average, all trucks and combinations	28	20	11	4	1	(1)
Mountain:						
2-axle, 6-tire	—	—	—	—	—	—
3-axle	81	55	43	17	4	—
Average, single-unit trucks	3	2	2	1	(1)	—
Truck-tractor and semitrailer	221	159	102	32	9	1
Truck and trailer	262	196	126	31	(1)	—
Average, truck combinations	230	167	107	32	7	1
Average, all trucks and combinations	54	39	25	8	2	1
Pacific:						
2-axle, 6-tire	—	—	—	—	—	—
3-axle	12	2	—	—	—	—
Average, single-unit trucks	1	(1)	—	—	—	—
Truck-tractor and semitrailer	239	186	119	33	9	3
Truck and trailer	369	279	167	17	7	4
Average, truck combinations	275	212	132	29	8	3
Average, all trucks and combinations	91	70	43	10	3	1
United States average:						
2-axle, 6-tire	(1)	(1)	(1)	—	—	—
3-axle	67	50	31	14	8	2
Average, single-unit trucks	2	2	1	(1)	(1)	(1)
Truck-tractor and semitrailer	118	79	45	15	5	1
Truck and trailer	302	247	172	52	19	5
Average, truck combinations	131	91	54	18	6	1
Average, all trucks and combinations	41	29	17	5	2	(1)
Comparative average, 1951	39	26	17	5	2	(1)
Comparative average, 1950	44	33	22	8	3	1

[1] Less than 5 per 10,000.

Pacific, Mountain, and East North Central regions than elsewhere, whereas table 9 shows that these regions are among those having the lowest frequencies of heavy axle loads. The far more frequent use of multiple-axle vehicles in these areas enables vehicle operators to distribute better their heavy loads. This is particularly true in California and other nearby States.

In considering the data concerning the frequencies of vehicles exceeding the State legal limits and the A.A.S.H.O. recommendations, especially the frequencies in the Middle Atlantic and New England regions, the fact should be recognized that higher limits generally are permitted under State laws in these areas than are recommended by the Association. Axles exceeding the recommended limits by as much as 25 percent may be within the legal limits of certain States, particularly in these two regions. Some States have no axle-group limits and one State has no prescribed axle-load limit in their motor-vehicle restrictions, a fact that further complicates direct comparison of excess weights based on law and those based on the recommendations. Comparison of the frequency data for New England and the Middle Atlantic regions shows that only about one-half of the vehicles exceeding one or more of the Association's recommendations actually exceeded a State legal limit. Due to more stringent weight laws in the East South Central and in the West North Central regions, the frequencies of vehicles exceeding the weight limits in the States of these areas are slightly greater than the frequencies of those exceeding the Association's recommendations. In the West South Central region, the frequencies of vehicles exceeding the State weight laws and of those exceeding the Association's recommendations are the same. For the United States as a whole, over two-thirds of the vehicles exceeding one or more of the Association's recommendations also exceeded a State legal limit.

Influence of Population, Sales, and Employment on Parking

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH BUREAU OF PUBLIC ROADS

Reported by S. T. HITCHCOCK
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The principal objective of traffic studies of any type is to obtain information needed for planning the improvement of highway facilities. An auxiliary objective of traffic studies is to improve the methods of making the studies, not only with respect to accuracy or completeness of the information, but also with respect to costs and time. A disturbing circumstance in making traffic studies is the length of time it takes to appraise a problem situation, obtain the information for solving the problem, analyze the data, and present the facts and recommendations for programming improvements.

The purpose of this discussion is to point out a few empirical relations which exist between certain basic economic factors such as population, retail sales, employment in urban areas, and the parking of automobiles, one part of the traffic problem which has been studied in the downtown area of many cities. If these relations can be shown to exist, it should be possible to learn how to use them in making parking and traffic studies in a few weeks instead of months and with considerable less expense.

Parking studies are one form of traffic studies, limited in most instances to problem situations in the downtown areas of cities where the economic interests of the community are centered and where traffic and transportation is focused to serve those interests. The increased use of the private automobile has made this problem one of common concern in all cities throughout the nation. The terminal problem in the downtown area is the concern not only of business in the area, but also of several departments of city government charged with the responsibilities of highway or street construction and maintenance, of traffic regulation, and of raising money for the administration of city government.

PRIOR to 1945, parking studies of limited area had been undertaken with expedient methods but without effort of a comprehensive nature or for the entire business district. In 1945, the first comprehensive type studies,[1] which now number 76, were initiated. Of the 76 parking studies, reports are available for analysis from 58 cities (table 1). These studies are centered around the people who drive their cars to the downtown area, and information is obtained about where they parked, how long they stayed, where they went after parking, and for what purpose. The reports from these studies are proving to be extremely useful in the planning and development of local parking improvements. Because of the similarity of methods used in obtaining information in these 58 cities, the reports have much added value in permitting a comparison of conditions and data for groups of these cities having similar characteristics.

Many cities are generally recognized as being of a certain type such as a manufacturing city, a retail market center, a resort town, or a center of governmental activity. Different economic factors have been used from time to time by market analysts and by other economists to express these differences. Several series of statistics that are available for this comparison provide an indication of the economic classification of cities. These are (1) census of population, 1950, (2) census of manufactures, 1947, (3) census of business, retail, wholesale, and service trades, 1948, and (4) economic classification of cities.[2]

For several years the International City Managers' Association has published in the Municipal Yearbook a classification of cities based primarily on retail and manufacturing employment characteristics for cities in different population groupings. These factors and several others generally

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[1] Proceedings of the Highway Research Board, vol. 25, 1945, p. 269, and vol. 26, 1946, pp. 430-444.

[2] Municipal Yearbook, 1950, p. 48.

DEFINITIONS

Average parking accumulation.—The number of vehicles parked at a particular time is the accumulation at that time. The average parking accumulation is the average of the volumes parked at each one-half-hour interval from 10 a.m. to 6 p.m.

Employment in manufactures.—Number of all employees (average for the year) for each urban place in each State. *Census of manufactures*, Bureau of the Census, vol. III, table 2, 1947.

Employment per 100 population.—The number of persons who work in a city per 100 inhabitants. It includes employment in manufacturing and retail, wholesale, and service trades.

Employment in retail trade.—Number of paid employees, work week ended nearest November 15, 1948, for each city in each State. *Census of business*, Bureau of the Census, vol. III, table 103, 1948. Retail trade-area statistics.

Employment in service trade.—Number of paid employees, work week ended nearest November 15, 1948, for each city in each State. *Census of business*, Bureau of the Census, vol. VII, tables 103A, B, and C, 1948. Service trade-area statistics. Service trade includes automotive repair, laundry, cleaning, amusements, hotels, and restaurants.

Employment in wholesale trade.—Number of paid employees, work week ended nearest November 15, 1948, for each city in each State. *Census of business*, Bureau of the Census, vol. V, table 103, 1948. Wholesale trade-area statistics.

G.A.F. retail sales.—The dollar volume of annual retail sales of general merchandise (*G*), apparel (*A*), and furniture, appliance, and furnishings (*F*) as reported in table 103 for each State. *Census of business*, Bureau of the Census, vol. III, 1948. Retail trade-area statistics.

Number of parkers.—This is the number of vehicles parked in the central business district at any time between 10 a.m. and 6 p.m. at the curb and offstreet, legal and illegal, and it includes trucks as well as passenger vehicles. All studies used in this analysis were made under comparable methods. In some instances modification was necessary to develop volumes on a 10 a.m. to 6 p.m. 8-hour period.

Rank-size.—A listing of cities ranked in order of size from largest to smallest.

Retail trade-manufacturing employment ratio.—Ratio of employment in retail trade business to employment in manufacturing industry.

Urban population.—Number of inhabitants of urbanized area if 50,000 persons or more; places under 50,000 population is the city population. Tables 18 and 24 P-A1, *Census of population*, Bureau of the Census, 1950.

Vehicle cordon count.—A count of traffic crossing the boundary of the survey area.

Table 1.—Cities in which comprehensive parking studies have been made

City and State	Year of study	1950 population of urbanized area
Albert Lea, Minn.[1]	1947	13,545
Albuquerque, N. Mex.[2]	1949	96,815
Alexandria, La.	1947	34,913
Allentown, Pa.[3]	1948	106,756
Anderson, Ind.	1948	46,820
Anderson, S. C.	1947	19,770
Atlanta, Ga.[1]	1945	507,887
Baltimore, Md.	1946	1,161,852
Bethlehem, Pa.[1] [3]	1948	66,340
Biddeford-Saco, Maine[1]	1950	31,160
Boise, Idaho	1948	34,393
Bristol, Va.–Tenn.[1]	1950	32,725
Butler, Pa.[1]	1951	23,482
Charlotte, N. C.	1947	140,930
Chattanooga, Tenn.	1947	167,764
Cleveland, Ohio	1948	1,383,599
Clovis, N. Mex.	1950	17,318
Coatesville, Pa.[1]	1951	13,826
Columbus, Ind.	1948	18,370
Corpus Christi, Tex.	1947	122,956
Dallas, Texas	1950	538,924
Decatur, Ind.	1948	7,271
Easton, Pa.	1948	35,632
Eugene, Oreg.[1]	1952	35,879
Evansville, Ind.	1949	137,573
Fond du Lac, Wis.[1]	1950	29,936
Frankfort, Ind.	1948	15,028
Gary, Ind.[3]	1949	133,911
Harrisburg, Pa.	1946	169,646
Honolulu, T. H.[1] [2]	1947	248,034
Huntington, Ind.	1948	15,079
Independence, Mo.[1]	1950	36,963
Jacksonville, Fla.	1947	242,909
Knoxville, Tenn.	1946	148,166
Kokomo, Ind.	1948	38,672
Lake Charles, La.	1947	41,272
Lexington, Ky.[1] [2]	1952	55,534
Lincoln, Nebr.	1950	99,509
Louisville, Ky.	1951	472,736
Lynchburg, Va.	1948	47,727
Martinsville, Va.	1949	17,251
Meadville, Pa.	1948	18,972
Memphis, Tenn.	1950	406,034
Miami, Fla.	1951	458,647
Monroe, La.	1947	38,572
Morristown, N. J.[1]	1950	17,124
Nashville, Tenn.[1]	1946	258,887
New Haven, Conn.	1946	244,836
Norristown, Pa.	1949	38,126
Ogden, Utah [1]	1952	57,112
Omaha, Nebr.	1948	310,291
Pawtucket, R. I.[3]	1945	81,436
Portland, Oreg.	1946	512,643
Portsmouth, N. H.	1946	18,830
Pottstown, Pa.	1949	22,589
Providence, R .I.	1945	583,346
Reading, Pa.	1947	154,931
Reno, Nev.	1949	32,497
Richmond, Va.	1948	257,995
Roswell, N. Mex.	1950	25,738
Rushville, Ind.[1]	1950	6,761
St. Louis, Mo.	1950	1,400,058
Seattle, Wash.	1946	621,509
Seymour, Ind.	1948	9,629
Spokane, Wash.	1947	176,004
Steubenville, Ohio [1]	1952	35,872
Stevens Point, Wis.	1947	16,564
Syracuse, N. Y.	1951	265,286
Toledo, Ohio	1947	364,344
Topeka, Kans.	1951	89,104
Uniontown, Pa.	1950	20,471
Wabash, Ind.	1948	10,621
Walla Walla, Wash.	1946	24,102
Waynesboro, Va.[1]	1952	12,357
West Chester, Pa.	1949	15,168
Wichita, Kans.	1947	194,047

[1] Cities not included in this study because their report was not received in time, did not provide sufficient detailed information, or data were not comparable to the 58 cities studied because of unusual conditions in the area.

[2] City population only.

[3] Part of an urbanized area. Population shown is that of named city only.

used were listed in this analysis for each of the 58 cities. As a result of an examination of these factors, it appeared that there were five groupings which could be readily recognized. Parking volumes for cities in each group were then studied in relation to the economic characteristics of the same group.

Several approaches were made to compare parking habits and volumes with economic characteristics of cities, three of which appear to offer useful methods of estimating parking volumes:

1. Rank-size order. Cities in which the parking studies had been made were arranged in order of size with respect to population, retail sales of general merchandise, apparel, and furniture (commonly known as G.A.F. sales), employment in manufacturing, and parking.

2. G.A.F. sales and parking. The number of parkers per million dollars of G.A.F. sales was compared with urban population.[3]

3. Statistical evaluation. The standard error of estimate was determined for 17 different factors in 31 different combinations.

Conclusions

Based on these studies the following conclusions are warranted:

1. Cities of similar economic characteristics may be recognized by types of employment and by population change, and may be grouped according to ratios based on these factors.
2. The amount of parking in the central business district has an empirical relation to the overall economy of the community.
3. Patterns of parking in a rank-size order have a definite relation to rank-size of cities of similar characteristics as grouped above with respect to population, G.A.F. sales, and employment in manufacturing.
4. The volume of parking per one million dollars of G.A.F. sales has a definite relation to population.
5. A statistically reliable relation exists between parking volumes and certain independent variables, and for certain groups of cities an equation can be developed to express this relation.
6. Estimates made by rank-size order, by the relation of parking volumes to G.A.F. sales, and by statistical evaluation when compared with actual observed volumes indicate that the volume of parkers for the central business district, as a whole, can be estimated within reasonable limits of accuracy.
7. Additional studies should be made to demonstrate how these relations can be used in planning parking facility improvements either directly in relation to land use or in combination with other known relations involving available parking spaces, trip purpose, length of time parked, and distance walked.

Economic Classification

The factors which were found to be usable in grouping the cities according to economic conditions were (1) retail trade-manufacturing employment ratio, which is the ratio of the number of persons employed in retail trade to the number employed in manufacturing; (2) employment per 100 population; (3) percent of population increase for the period 1940–50; and (4) percent of employment in service trades, which is the percent of total employment in such establishments as amusements, laundries, dry cleaning places, and hotels. These factors are summarized in table 2.

Employment in wholesale trade, motor-vehicle registrations in urban areas, total retail sales, and gasoline filling station sales were among other factors considered. They either duplicated trends apparent in the four factors selected, were not available for some cities, or were too far out of date to be used with assurance.

A considerable range in economic activity is represented in this group of cities. Employment in retail trade and manufacturing together account for more than one-half the total employment in all 58 cities and more than 80 percent in 31 of the 58 cities. The distribution of employment between these two factors, however, varies from five persons employed in retail trade for every person in manufacturing in Clovis, New Mex., to one person in retail trade for every six persons in manufacturing in Anderson, Ind. (table 2).

When cities are arrayed in order of the retail trade-manufacturing employment ratio, a progressive group classification can be made ranging from a group dominant in retail trade with a ratio of 1:0.2 through cities of decreasing retail trade importance to a group dominant in manufacturing with a ratio of 1:4.2 (table 2). Group averages for employment per 100 population arranged in the same group sequence increases from 15 to 36 percent. Employment in service trades tends to increase with dominance of retail trade and, generally speaking, population growth tends to be greatest in the cities showing greatest dominance in retail trade.

The population increase of 13.2 percent in group V cities is apparently inconsistent with the trend evident in the other groups and in relation to employment ratios. One of the cities in this group, Columbus, Ind., is located near Camp Atterbury, and the exceptional population growth of 56.5 percent in the 10-year period 1940–50 may be due to this proximity. Wabash, Seymour, and Decatur, all in Indiana, are also in this group and show population growth contrary to the apparent trend. These cities and Columbus, according to the census of manufactures, show increased industrial activity from 50 to 120 percent in production workers in the 1937–47 period, whereas other cities in this group show increases of less than 25 percent.

Individual cities vary somewhat from these group averages as might be expected, and 8 of the 58 cities do not seem to fit any of the 5 groups. Whether other or additional groupings would be significant if more cities could be included is conjectural.

Group I Cities

Retail trade-manufacturing employment ratio ranges from 1:0.2 to 1:0.4 with an average of 1:0.23 for the eight cities in this group. Employment in manufacturing is less than 20 percent.

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[3] *Gross retail sales and automobile parking requirements* by Floyd M. Jennings, Highway Research Board, Bulletin No. 19, 1948.

Table 2.—Population and employment in 58 cities in which comparable parking studies have been made

City and State	Urbanized area population	City population increase, 1940–50	Employment per 100 population of city	Persons employed in— Manufacturing		Retail trade		Wholesale trade		Service trades		Total		Retail trade-manufacturing employment ratio
				Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution	
Group I Cities														
	Thousands	*Percent*		*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	
Miami, Fla.	458.6	44.8	19.0	7.1	15.0	22.5	47.5	8.4	17.8	9.3	19.7	47.3	100.0	1:0.3
Corpus Christi, Tex.	122.9	89.0	12.2	2.6	17.6	7.2	47.7	2.5	17.0	2.7	17.7	15.0	100.0	1:0.4
Albuquerque, N. Mex.	[1] 96.8	173.1	12.2	1.5	12.4	6.4	53.7	1.9	16.3	2.1	17.6	11.9	100.0	1:0.2
Lake Charles, La.	41.3	94.6	11.5	0.6	13.7	2.5	53.2	0.8	16.5	0.8	16.6	4.7	100.0	1:0.2
Boise, Idaho	34.4	31.6	19.8	0.8	11.8	3.5	50.9	1.4	20.2	1.1	17.1	6.8	100.0	1:0.2
Reno, Nev.	32.5	52.4	24.9	0.7	9.0	3.5	42.9	0.8	9.6	3.1	38.5	8.1	100.0	1:0.2
Roswell, N. Mex.	25.7	90.9	10.8	0.3	12.4	1.7	59.4	0.3	10.4	0.5	17.8	2.8	100.0	1:0.2
Clovis, N. Mex.	17.3	72.1	11.0	0.2	8.8	1.1	55.8	0.3	16.9	0.3	18.5	1.9	100.0	1:0.2
Group II Cities														
Dallas, Tex.	538.9	47.4	25.3	34.2	31.2	37.0	33.7	25.2	22.9	13.5	12.2	109.9	100.0	1:0.9
Jacksonville, Fla.	242.9	18.2	21.1	12.2	28.3	15.0	34.9	9.4	21.8	6.5	15.0	43.1	100.0	1:0.8
Wichita, Kans.	194.0	46.4	17.0	9.6	29.1	12.9	39.1	5.7	17.3	4.8	14.5	33.0	100.0	1:0.7
Spokane, Wash.	176.0	32.6	16.0	6.5	25.2	10.5	40.6	5.4	20.8	3.5	13.4	25.9	100.0	1:0.6
Lincoln, Nebr.	99.5	20.6	19.6	7.3	37.4	8.0	41.3	2.0	10.1	2.2	11.2	19.5	100.0	1:0.9
Topeka, Kans.	89.1	16.2	15.7	4.8	34.0	5.8	41.5	1.7	12.2	1.7	12.3	14.0	100.0	1:0.8
Monroe, La.	38.6	36.3	20.0	1.7	22.8	3.3	42.3	1.4	17.8	1.3	17.1	7.7	100.0	1:0.5
Alexandria, La.	34.9	29.0	17.6	1.6	26.1	2.7	44.7	0.8	12.2	1.0	17.0	6.1	100.0	1:0.6
Walla Walla, Wash.	24.1	33.0	19.0	1.2	25.3	2.1	46.0	0.7	14.5	0.6	14.2	4.6	100.0	1:0.6
Group III Cities														
Baltimore, Md.	1,161.8	10.5	25.0	120.9	51.0	69.3	29.2	26.2	11.0	20.8	8.8	237.2	100.0	1:1.7
Seattle, Wash.	621.5	27.0	24.8	50.2	43.5	34.2	29.5	20.0	17.3	11.3	9.7	115.7	100.0	1:1.5
Portland, Oreg.	512.6	22.3	24.1	33.8	37.5	30.5	33.9	15.8	17.5	10.0	11.1	90.1	100.0	1:1.1
Memphis, Tenn.	406.0	35.2	21.1	33.2	38.7	27.6	32.3	15.1	17.6	9.7	11.4	85.6	100.0	1:1.2
Omaha, Nebr.	310.3	12.2	24.6	25.4	41.2	17.7	28.8	11.7	19.0	6.8	11.0	61.6	100.0	1:1.4
Richmond, Va.	258.0	19.3	29.3	28.0	41.5	22.2	32.8	11.0	16.2	6.4	9.5	67.6	100.0	1:1.3
Knoxville, Tenn.	148.2	11.8	26.4	19.3	49.3	11.4	29.2	4.5	11.5	3.9	10.0	39.1	100.0	1:1.7
Charlotte, N. C.	140.9	32.8	27.0	13.5	37.3	10.6	29.3	8.1	22.4	4.0	11.0	36.2	100.0	1:1.3
Portsmouth, N. H.	18.8	27.0	18.0	[2] 1.4	41.9	1.3	39.5	0.3	8.6	0.4	10.0	3.4	100.0	1:1.1
West Chester, Pa.	15.2	14.1	22.9	1.8	51.1	1.4	39.6	0.1	2.2	0.2	7.1	3.5	100.0	1:1.3
Frankfort, Ind.	15.0	9.6	16.2	1.2	48.7	1.0	41.8	0.1	3.6	0.1	5.9	2.4	100.0	1:1.2
Group IV Cities														
St. Louis, Mo.	1,400.0	5.0	36.2	173.0	55.8	67.6	21.8	45.1	14.6	24.3	7.8	310.0	100.0	1:2.6
Louisville, Ky.	472.7	15.7	28.2	55.6	53.7	24.4	23.6	13.6	13.0	10.0	9.7	103.6	100.0	1:2.3
Toledo, Ohio	364.3	7.5	36.8	59.5	61.6	22.4	23.2	8.2	8.5	6.5	6.7	96.6	100.0	1:2.7
Syracuse, N. Y.	265.3	7.1	30.2	35.3	53.0	18.1	27.3	7.6	11.4	5.5	8.3	66.5	100.0	1:2.0
Chattanooga, Tenn.	167.8	2.2	37.2	30.6	62.6	10.5	21.6	3.7	7.5	4.0	8.3	48.8	100.0	1:2.9
Allentown, Pa.	[1] 106.8	10.2	36.2	24.2	62.5	9.4	24.3	2.9	7.5	2.2	5.7	38.7	100.0	1:2.6
Lynchburg, Va.	47.7	7.2	31.9	8.8	57.8	4.0	26.1	1.1	7.3	1.3	8.8	15.2	100.0	1:2.2
Easton, Pa.	35.6	6.1	35.1	7.3	58.4	3.7	29.5	0.6	5.0	0.9	7.1	12.5	100.0	1:2.0
Huntington, Ind.	15.0	8.5	29.7	3.1	69.0	1.1	24.5	0.2	3.5	0.1	3.0	4.5	100.0	1:2.8
Group V Cities														
Cleveland, Ohio	1,383.6	4.2	38.8	223.6	63.1	68.9	19.5	36.9	10.4	24.9	7.0	354.3	100.0	1:3.2
Providence, R. I.	583.3	−1.9	16.3	61.2	64.4	19.6	20.7	8.1	8.5	6.1	6.4	95.0	100.0	1:3.1
New Haven, Conn.	244.8	2.4	22.7	33.5	60.2	11.1	20.0	6.7	12.0	4.3	7.8	55.6	100.0	1:3.0
Reading, Pa.	154.9	−1.1	26.6	27.1	65.8	9.0	21.8	2.5	6.0	2.6	6.4	41.2	100.0	1:3.0
Gary, Ind.	[1] 133.9	19.9	34.6	35.6	76.8	7.5	16.3	1.1	2.4	2.1	4.5	46.3	100.0	1:4.7
Pawtucket, R. I.	[1] 81.4	7.4	34.1	20.9	75.4	4.9	17.6	0.8	3.1	1.1	3.9	27.7	100.0	1:4.3
Anderson, Ind.	46.8	12.6	27.1	22.9	82.0	3.6	13.1	0.5	1.7	0.9	3.2	27.9	100.0	1:6.4
Kokomo, Ind.	38.7	14.4	38.0	11.1	75.5	2.7	18.2	0.4	2.6	0.5	3.7	14.7	100.0	1:4.1
Pottstown, Pa.	22.6	11.9	50.3	8.9	78.3	1.8	16.1	0.4	3.1	0.3	2.5	11.4	100.0	1:4.9
Columbus, Ind.	18.4	56.5	44.4	6.0	73.2	1.6	19.0	0.4	5.0	0.2	2.8	8.2	100.0	1:3.8
Wabash, Ind.	10.6	10.0	49.9	[3] 4.3	82.2	0.8	14.6	0.1	2.0	0.1	1.2	5.3	100.0	1:5.4
Seymour, Ind.	9.6	11.7	46.7	3.3	73.9	0.9	20.0	0.1	2.8	0.1	3.3	4.4	100.0	1:3.7
Decatur, Ind.	7.3	24.1	44.1	2.5	76.9	0.5	16.8	0.1	4.1	0.1	2.2	3.2	100.0	1:5.0
Cities not Readily Classified by Groups														
Harrisburg, Pa.	169.6	6.7	14.6	8.7	35.1	9.2	37.2	4.1	16.6	2.7	11.1	24.7	100.0	1:0.9
Evansville, Ind.	137.6	32.5	29.1	25.2	63.0	8.8	21.9	3.3	8.2	2.8	6.9	40.1	100.0	1:2.9
Norristown, Pa.	38.1	−0.1	19.8	4.1	54.9	2.5	32.7	0.4	5.3	0.5	7.1	7.5	100.0	1:1.6
Uniontown, Pa.	20.5	0.3	32.5	2.1	31.6	3.3	48.5	0.7	10.8	0.6	9.1	6.7	100.0	1:0.6
Anderson, S. C.	19.8	1.8	27.1	2.4	44.5	2.2	40.6	0.3	5.9	0.5	9.0	5.4	100.0	1:1.1
Meadville, Pa.	19.0	−6.2	20.5	1.4	36.9	1.9	47.9	0.2	4.4	0.4	10.8	3.9	100.0	1:0.7
Martinsville, Va.	17.2	71.1	24.7	2.6	59.5	1.2	28.6	0.1	3.2	0.4	8.7	4.3	100.0	1:2.2
Stevens Point, Wis.	16.6	5.0	20.3	1.7	49.9	1.1	31.4	0.3	9.6	0.3	9.1	3.4	100.0	1:1.5
Summary														
Group I		81.1	15.2	1.7	12.6	6.1	51.4	2.0	15.6	2.5	20.4	12.3	100.0	1:0.2
Group II		31.1	19.0	8.8	28.8	10.8	40.5	5.8	16.6	3.9	14.1	29.3	100.0	1:0.7
Group III		20.2	23.6	29.9	43.8	20.6	33.3	10.3	13.3	6.7	9.6	67.5	100.0	1:1.3
Group IV		7.7	33.5	44.2	59.4	17.9	24.6	9.2	8.7	6.1	7.3	77.4	100.0	1:2.5
Group V		13.2	36.4	35.5	72.9	10.2	18.0	4.5	4.9	3.3	4.2	53.5	100.0	1:4.2

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[1] City population only.

[2] Modified to include an estimate for employment in nearby U. S. Navy Yard.

[3] County figure. Urban place employment volume is not available.

Employment per 100 population varies from 10.8 to 24.9 and averages 15.2. This group also includes the fastest growing cities of all the cities studied. Population increase during the 1940–50 period averaged 81.1 percent (table 2).

Economic activity in this group of cities is dominantly in retail trade with a much larger proportion of employment (20.4 percent) in service trades—hotels, service stations, amusements, and laundries, cleaning, and clothes repair establishments—than in the average city which is seldom more than 12 percent. This high percentage is an indication of resort centers and higher than average auto usage which is synonymous with resorts. Six of these eight cities are located in such areas.

Group II Cities

Retail trade is dominant in the economy of the cities in this group but more diversification in employment is evidenced. Employment in retail trade is slightly greater than in manufacturing. Employment in manufacturing exceeds 20 percent. Employment in both retail trade and manufacturing comprises about two-thirds of all employment covered by the censuses. Employment in service trades is 14.1 percent, only a little more than the average of all 58 cities. The retail trade-manufacturing employment ratio ranges from 1:0.5 to 1:0.9. Employment per 100 population varies from 15.7 to 25.3 with an average of 19.0.

Population increase averages 31.1 percent for the 10-year period 1940–50 with the newer cities showing more increase than the cities which were well established before the advent of the automobile.

Group III Cities

Economic activity in this group of cities may be considered to be nearly balanced, at least with respect to employment in retail trade and in manufacturing. Employment in manufacturing is the greater of the two but less than 50 percent of the total. The retail trade-manufacturing employment ratio ranges from 1:1.1 to 1:1.7 with an average of 1:1.3. Employment per 100 population varies from 16.2 to 29.3 and averages 23.6. Employment in service trades is generally 11 percent or less.

Population increase averaged 20.2 percent during the 1940–50 period. Here again may be noted the differences in pattern of cities which have experienced the greater portion of their growth since the advent of the automobile, and those which were fully developed as cities before the automobile became a means of transportation.

Group IV Cities

Economic activity in these cities is dominantly in manufacturing, although employment in retail trade represents nearly one-third of the employment in some instances. The retail trade-manufacturing employment ratio ranges from 1:2.0 to 1:2.9, and averages 1:2.5 for the group as a whole. Employment in retail trade is more than 20 percent. Employment per 100 population varies from 28.2 to 37.2 and averages 33.5. Employment in service trades is low, 7.3 percent of all employment.

Population increase in this group, on the average, is less than that in the first three groups and averages 7.7 percent for the group as a whole. This is somewhat less than the average for all cities in the country. No "new" cities are represented in this group. All were established urban communities before the development of the automobile, although all have shown some growth during this period.

Group V Cities

The economic activity in this group of cities is dominantly industrial in character, as is indicated by the relatively high proportion of employment in manufacturing in comparison with that in retail trade. The retail trade-manufacturing employment ratio ranges from 1:3.0 to 1:6.4 and averages 1:4.2. Employment per 100 population varies from a low of 16.3 to a high of 50.3 and averages 36.4. Employment in service trades is low, averaging 4.2 percent of all employment.

Rank-size Rule

After having established these groupings of cities by recognizing differences in certain economic characteristics, it should be possible to determine whether the relative amount of parking differs for these different groups; and if so, how it differs, and if these differences can be estimated.

The rank-size rule suggests one method for making such a determination. Economists for many years have recognized an empirical relation which exists in the distribution of urban population in large continental areas.[4] It was first recognized in Europe where it appeared that the economy of cities was somehow interconnected in spite of national boundaries. Regardless of change in population over the years, if all cities are arranged in order of population for any census period, the ratio of the population of any one city to the population of the largest city is approximately the same as its rank.

The rank-size rule means that the 10th largest city is about one-tenth the size of the largest city, and the 100th largest city, one-hundredth the size of the largest city. In the United States, for example, the urbanized area of New York has a population of 12.30 million persons. According to this theory the 10th largest city should have a population of about 1.23 million. Actually Cleveland, the 10th largest urbanized area, had a population of 1.38 million persons in 1950. The 100th largest city, the Bristol-New Britain urbanized area in Connecticut, had a population of 123,079, almost exactly one-hundredth the size of New York.

This relation[5] has been stated as $r.P^q = K$ where r = rank of a particular city in population.

P = population of the particular city.

q = constant (in the United States = 1 approximately).

K = constant for largest city population in the group (in the United States this would be New York where $r = 1$).

With the population of an individual city so closely determined by its rank in relation to all cities, it would appear reasonable to assume that the economy of each city might also have some similar relation to that of the other cities, and further that communication and transportation might also reflect some relation to this economic activity in the interchange of goods and persons. Inasmuch as the stores and office buildings where much of this interchange takes place are usually clustered in the downtown area of most cities, it is reasonable to study those factors which are centered there or which are dependent on that area as a center of a much larger trading area.

Factors Used in Ranking

These 58 cities were ranked in four different ways: (1) in order of urban population, (2) in order of volume of retail sales of general merchandise, apparel, and furniture (G.A.F. sales), (3) in order of employment in manufacturing, and (4) in order of number of parkers. These are detailed in table 3. Other factors involving employment in retail trade, motor-vehicle ownership, and gasoline service station sales were also tested, but were discarded since it appeared that they duplicated one of the other rankings or were incomplete for some of the cities.

The purpose of making these rankings is to determine if similar proportional relations might exist with respect to parking volumes as exist with respect to population, or if some patterns in ranked position might be typical of certain economic characteristics.

Information for establishing proportional relations with respect to parking volumes proved to be inadequate because the largest parking volume and the city having the largest parking volume are unknown, and because 58 studies are too few to establish a ranking representative of all cities in the United States. Fifty-eight cities represent but 2.4 percent of all cities of 5,000 population or more.

When the rankings of these factors for each city were compared, five fairly distinct patterns were apparent if the pattern with respect to parking is omitted. Examination of the cities in each of these five patterns revealed the fact that, with but a few exceptions, the cities were the same as those grouped according to similarity of economic factors. Accordingly, a pattern representing the averaged ranked positions for each group was developed as being characteristic of that group. Of the remaining eight cities four have patterns similar to one of the five ranked patterns, but not having economic characteristics classifiable in the five groups, they have been omitted from further comparisons. The remaining four have some eccentricity which does not seem to fit any of the five patterns. After having recognized these general patterns, the ranked characteristics of the factors including parking are discussed for each of the economic groups.

Group I Cities

The characteristic pattern of the rankings of the four factors in the eight cities in group I finds the ranked position of urban population and G.A.F. sales almost the same, employment in

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[4] Christaller, Loesch, Stouffer, Zipf, Isard, among others.

[5] Quarterly Journal of Economics, Harvard University, May 1951.

Table 4.—Ranked position with respect to urban population, G.A.F. sales, employment in manufacturing, and parking in 58 cities in which comparable studies have been made

City and State	Ranked position in—			
	Urbanized area population	G.A.F. retail sales	Employment in manufacturing	Number of parkers
St. Louis, Mo.	1	3	2	1
Cleveland, Ohio	2	1	1	2
Baltimore, Md.	3	2	3	8
Seattle, Wash	4	5	7	6
Providence, R. I.	5	12	4	18
Dallas, Tex.	6	4	10	3
Portland, Oreg.	7	6	11	4
Louisville, Ky.	8	8	6	9
Miami, Fla.	9	11	32	7
Memphis, Tenn.	10	7	13	17
Toledo, Ohio	11	9	5	11
Omaha, Nebr.	12	13	17	14
Syracuse, N. Y.	13	14	9	19
Richmond, Va.	14	10	15	10
New Haven, Conn.	15	16	12	28
Jacksonville, Fla.	16	15	24	12
Wichita, Kans.	17	18	26	5
Spokane, Wash.	18	19	33	16
Harrisburg, Pa.	19	21	29	26
Chattanooga, Tenn.	20	23	14	24
Reading, Pa.	21	24	16	29
Knoxville, Tenn.	22	17	22	37
Charlotte, N. C.	23	22	23	21
Evansville, Ind.	24	26	18	20
Gary, Ind.	25	25	8	32
Corpus Christi, Tex.	26	28	40	27
Allentown, Pa.	27	20	19	30
Lincoln, Nebr.	28	27	30	15
Albuquerque, N. Mex.	29	29	49	22
Topeka, Kans.	30	30	35	13
Pawtucket, R. I.	31	31	21	40
Lynchburg, Va.	32	37	28	31
Anderson, Ind.	33	38	20	49
Lake Charles, La.	34	41	55	44
Kokomo, Ind.	35	42	25	46
Monroe, La.	36	34	45	45
Norristown, Pa.	37	40	37	38
Easton, Pa.	38	32	31	34
Alexandria, La.	39	39	48	47
Boise, Idaho	40	33	54	25
Reno, Nev.	41	36	56	23
Roswell, N. Mex.	42	47	57	33
Walla Walla, Wash.	43	44	52	41
Pottstown, Pa.	44	45	27	39
Uniontown, Pa.	45	35	44	35
Anderson, S. C.	46	43	43	36
Meadville, Pa.	47	46	50	43
Portsmouth, N. H.	48	53	51	56
Columbus, Ind.	49	51	34	48
Clovis, N. Mex.	50	52	58	42
Martinsville, Va.	51	48	41	54
Stevens Point, Wis.	52	50	47	53
West Chester, Pa.	53	49	46	50
Huntington, Ind.	54	54	39	51
Frankfort, Ind.	55	55	53	55
Wabash, Ind.	56	57	36	52
Seymour, Ind.	57	56	38	57
Decatur, Ind.	58	58	42	58

manufacturing considerably lower, and parking in a higher ranking position. For the group as a whole, the average city ranks about 17 places lower in employment in manufacturing than in population, while parking ranks 8 places higher than population (fig. 1). These differentials are not so great in the larger cities probably for the reason that fewer cities are represented in the upper range of values, and hence there is less change in ranked position. The patterns with relation to employment in manufacturing, population, and G.A.F. sales are nearly all similar.

Relatively low income and consequently low motor-vehicle ownership, travel, and hence parking may be the influence which positions parking in Lake Charles in an opposite direction from that of other cities in this group. The ranked position of parking for Corpus Christi is not as high relatively as for the other cities in this group. It is a resort city but the 1:0.4 retail trade-manufacturing ratio is lower than the average for the group (table 3).

Group II Cities

The characteristic pattern of the rankings of the four factors for the nine cities in group II shows a similarity to group I, but with less differentials in rank between employment in manufacturing, parking, population, and G.A.F. sales. The average city in this group ranks about eight places lower in employment in manufacturing than in population, and about the same amount higher in parking (fig. 1). The patterns of the three economic factors other than parking are similar.

The rank of parking in Wichita, Lincoln, and Topeka with respect to population is greater than in other cities in this group—almost as much as for group I cities. These three cities have relatively wide streets and, with angle parking permitted at some curbs, have larger numbers of curb spaces available than in the average city. Lincoln and Topeka are both centers of government employment which is not included in the computation of employment ratios. With many government employees parking in the downtown area twice a day as they do in the cities of this size — going home for lunch — the number of parkers is duplicated to some extent. This could account for the increased rank position of these cities with respect to parking (table 3).

The ranked position of parking in Alexandria and Monroe, La., is lower than for other cities in this group. Low parking with respect to population and G.A.F. sales is characteristic of cities where the economy is dominated by manufacturing, but this is not the case in these two cities. It appears as in the case of Lake Charles that for the city as a whole, they are low income cities which means low purchasing power, low car ownership, and hence, low parking volumes.

Group III Cities

The characteristic pattern of the rankings of the four factors for 11 cities in group III shows the rankings to be approximately the same for all factors. There is a tendency for the differential in ranking to be less in the larger cities as in the other groups. For the group as a whole, there is less than one position variation in rank in any one of the factors with relation to any of the other factors (fig. 1). A spread of five or six positions between high and low rankings for the four factors is not significantly different for individual cities, particularly in the smaller cities where six cities lie between the 50th and 55th positions inclusive for population with a variation between 15,000 and 17,300 persons (table 3).

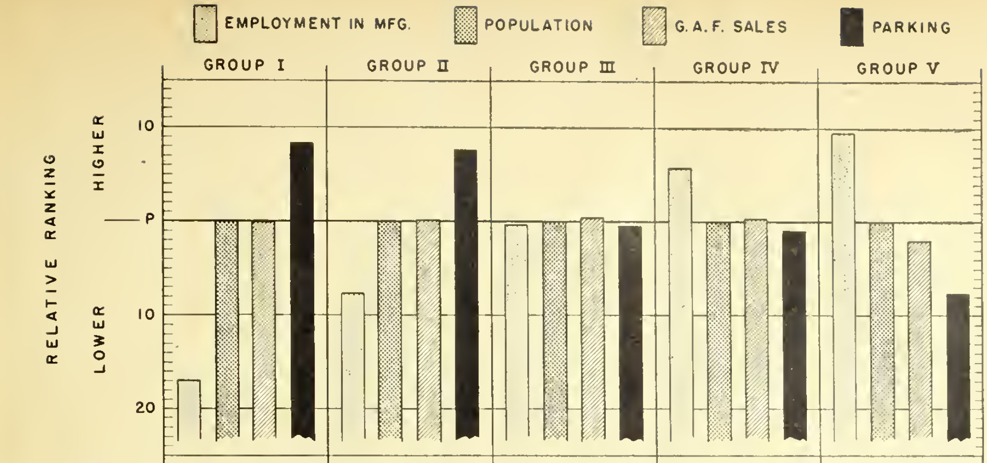
The ranked position of parking in Memphis, Baltimore, and Knoxville is somewhat less than for the group as a whole. In these cities, as in the three Louisiana cities, the proportion of nonwhite population is large, the purchasing power less than average, and hence automobile ownership and parking could be expected to be less, but all of the cities in this group have similar patterns with relation to employment in manufacturing, population, and G.A.F. sales.

Group IV Cities

The characteristic pattern of the rankings of the four factors for the nine cities in this group shows no appreciable difference as far as population, G.A.F. sales, and parking is concerned. Ranking with respect to employment in manufacturing for the group as a whole is about six positions higher than for other factors. The reason for the upswing in ranked positions for parking in Lynchburg is not apparent. It has similar patterns of rank with respect to the other three factors.

Group V Cities

The characteristic pattern of the rankings of the four factors for the 13 cities in this group shows a reversal in position with respect to the cities in group I. Generally speaking, the cities



	GROUP I	GROUP II	GROUP III	GROUP IV	GROUP V
NUMBER OF CITIES	8	9	11	9	13
RETAIL TRADE MFG. EMPLOYMENT RATIO	1:0.2	1:0.7	1:1.3	1:2.5	1:4.2
EMPLOYMENT PER 100 POPULATION	15.2	19.0	23.6	33.5	36.4
POPULATION INCREASE 1940-1950	81.1%	31.1%	20.2%	7.7%	13.2%
PERCENT EMPLOYMENT IN SERVICE TRADE	20.4%	14.1%	9.6%	7.3%	4.2%

Figure 1.—Relative rank of parking, employment in manufacturing, and G.A.F. sales with respect to population in five groups of cities in which comparable parking studies have been made.

in this group rank higher by about 12 positions in manufacturing than in population. Ranking in G.A.F. retail sales is about three positions lower than population, and parking is ahout five positions lower than that (fig. 1). Here again the differential in ranking is less in larger cities.

The patterns of the ranked positions of Pottstown, Pa., and Columbus and Wabash, Ind., show a relative higher ranking for parking than for the group as a whole. The actual differences in rank for these cities represent relatively small volumes of parkers. Pottstown, for example, although ranked 39th in parking with 10,600 parkers, is less than 10 percent greater than the 45th city having 9,600 parkers.

Where a given city in group I has a ranked position with respect to population of P, it will generally have a ranked position with respect to parking of $P - 8.3$ although some judgment should be used when the cities under consideration are at the extreme ends of the rankings (table 4). This is shown graphically in figure 1.

If the groupings which have been established were based only on rank-size patterns, Norristown, Pa., Martinsville, Va., Stevens Point, Wis., and Meadville, Pa., would also be included in one of the groups. Ranked position of the remaining four cities do not seem to fit any of the five patterns, nor are there enough studies involving patterns of their type to suggest more than five patterns of rank-size characteristics. Reasons for their eccentricities are not apparent.

Consideration of these rank-size patterns seems to confirm the groupings of cities made on a basis of economic classification and to indicate that with known ranked positions of population, G.A.F. sales, and employment in manufacturing, a definite relative ranked position in parking can be expected to follow. Further, it appears that with known parking volumes for each rank (fig. 2), the volume of parking can be estimated for cities in which parking studies have not yet been made, hut which can be properly grouped by use of the economic factors.

G.A.F. Sales, Population, and Parking

The second approach to this study is the relation of G.A.F. sales, population, and parking.[6] Kenneth C. Welch, vice president of the Grand Rapids Store Equipment Co., has studied the records of the Comptroller's Congress and of other agencies summarizing data relative to retail sales over a period of more than 20 years. As a result of these studies, a relation was developed between parking area requirements and the retail sales dollar. When this suhject was presented, hope was expressed that this relation could be given further study under a wider range of conditions and with more data. The completion of reports on parking studies in more than 50 cities provides quantitative information about parking volumes for the central business district. The census of business for 1948 provides information on the volume of retail sales in each city. Some of this volume accrues to stores which are not necessarily part of the downtown area and in many instances are not. Examples are food, lumber, gasoline service stations, and drug stores. Sales of general merchandise, apparel, and furniture, appliances and furnishings (G.A.F. sales) are generally downtown functions.

In this article it has been assumed that they are a measure of the retail sales in the entire downtown area. There are other functions in the downtown area which attract traffic and persons such as recreational facilities, professional services, etc., but from many points of view the downtown area may he considered as one big department store with the many auxiliary services needed to support the economy of the trading area of that big store. The underlying principle of this assumption is that parking requirements of a community are more closely allied to the volume of retail business than they are to any other single factor.

The thought inherent in this basis for estimating parking needs is that retail sales is a source of data that, when analyzed, provides a community with a true picture of parking requirements and not merely a picture of an existing parking pattern. This could be applied to the needs of large individual retail units or a group of units comprising even the entire central business district. In most cities of the size under consideration, retail sales of general merchandise, apparel, and furniture and furnishings (G.A.F. sales) are a central business district function. Other retail sales are generally in "convenience" locations near consumer residences or near railroad stations or sidings for bulky merchandise.

There are some nonretail land and structural uses in most central business districts, but the

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[6] See footnote 3, p. 249.

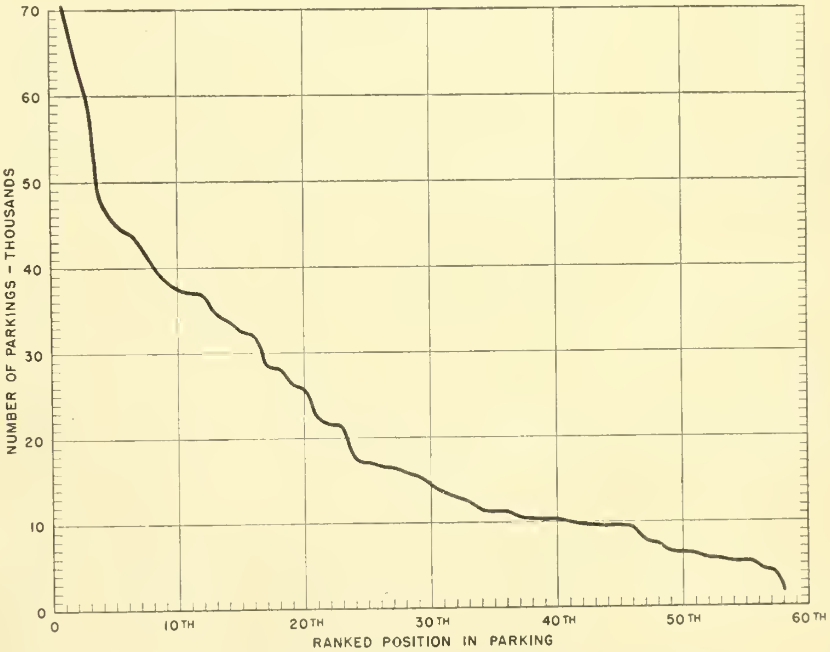

Figure 2.—Number of parkings and ranked position with respect to number of parkings in 58 cities in which comparable parking studies have been made.

Table 4.—Ranked position with respect to urban population, G.A.F. retail sales, employment in manufacturing, and parking in 58 cities in which comparable studies have been made

City and State	Ranked position [1] in—			
	Urbanized area population	G.A.F. retail sales	Employment in manufacturing	Numbers of parkers
Group I				
Miami, Fla.	9	11	32	7
Corpus Christi, Tex.	26	28	40	27
Albuquerque, N. Mex.	29	29	49	22
Lake Charles, La.	(34)	(41)	(55)	(44)
Boise, Idaho	40	33	54	25
Reno, Nev.	41	36	56	23
Roswell, N. Mex.	42	47	57	33
Clovis, N. Mex.	50	52	58	42
Group II				
Dallas, Tex.	6	4	10	3
Jacksonville, Fla.	16	15	24	12
Wichita, Kans.	17	18	26	5
Spokane, Wash.	18	19	33	16
Lincoln, Nebr.	28	27	30	15
Topeka, Kans.	30	30	35	13
Monroe, La.	(36)	(34)	(45)	(45)
Alexandria, La.	(39)	(39)	(48)	(47)
Walla Walla, Wash.	43	44	52	41
Group III				
Baltimore, Md.	3	2	3	8
Seattle, Wash.	4	5	7	6
Portland, Oreg.	7	6	11	4
Memphis, Tenn.	(10)	(7)	(13)	(17)
Omaha, Nebr.	12	13	17	14
Richmond, Va.	14	10	15	10
Knoxville, Tenn.	(22)	(17)	(22)	(37)
Charlotte, N. C.	23	22	23	21
Portsmouth, N. H.	48	53	51	56
West Chester, Pa.	53	49	46	50
Frankfort, Ind.	55	55	53	55
Group IV				
St. Louis, Mo.	1	3	2	1
Louisville, Ky.	8	8	6	9
Toledo, Ohio	11	9	5	11
Syracuse, N. Y.	13	14	9	19
Chattanooga, Tenn.	20	23	14	24
Allentown, Pa.	27	20	19	30
Lynchburg, Va.	(32)	(37)	(28)	(31)
Easton, Pa.	38	32	31	34
Huntington, Ind.	54	54	39	51
Group V				
Cleveland, Ohio	2	1	1	2
Providence, R. I.	5	12	4	18
New Haven, Conn.	15	16	12	28
Reading, Pa.	21	24	16	29
Gary, Ind.	25	25	8	32
Pawtucket, R. I.	31	31	21	40
Anderson, Ind.	33	38	20	49
Kokomo, Ind.	35	42	25	46
Pottstown, Pa.	(44)	(45)	(27)	(39)
Columbus, Ind.	(49)	(51)	(34)	(48)
Wabash, Ind.	(56)	(57)	(36)	(52)
Seymour, Ind.	57	56	38	57
Decatur, Ind.	58	58	42	58
Cities not Readily Classified				
Harrisburg, Pa.	19	21	29	26
Evansville, Ind.	24	26	18	20
Norristown, Pa.[2]	37	40	37	38
Uniontown, Pa.	45	35	44	35
Anderson, S. C.	46	43	43	36
Meadville, Pa.[2]	47	46	50	43
Martinsville, Va.[2]	51	48	41	54
Stevens Point, Wis.[2]	52	50	47	53
Relative Rank with Respect to Population				
Group I	P	P—0.1	P+16.7	P—8.3
Group II	P	P—0.1	P+ 7.6	P—7.6
Group III	P	P—0.4	P+ 0.6	P+0.5
Group IV	P	P+1.1	P— 6.1	P+1.0
Group V	P	P+2.1	P— 9.5	P+7.7

[1] Figures in parentheses were not used in determination of average relative rank.

[2] Classifiable with respect to rank position but not by employment ratios and population increase.

bulk of the total parking space requirements for the central business district are those required for retail sales and professional offices. Remaining uses are not major factors affecting the "total space requirements" for the entire district. Existing land-use patterns demonstrate that as much as 45 percent of the offstreet property is devoted to retail sales purposes, and on an area basis including upper floors an even greater proportion is so used. The use of gross sales to reflect parking requirements is thus suggested as a basis for estimate.

The development of this basis for estimating parking needs recognizes that the extremely large cities, those of a million population or more and even perhaps a few nearing that size, cannot possibly solve the problem at the present time under existing urban land patterns. But in cities ranging up to one-quarter or one-half million people, the size and proportions of the central business district area remain in a functional character in a limited area, and are not too large in relation to the parking problem to be solved. There are in the central business districts of these cities extensive areas of land lying in the submarginal areas commonly called blighted commercial land. These areas present an excellent opportunity for providing parking as well as recovering some lost tax values.

Classification of Retail Trade

The Bureau of the Census in its census of business classifies retail trade in three business groups: convenience goods, shopping goods, and all others.

Convenience goods include foods, drugs, general stores, and gasoline service stations comprising 36.5 percent of all retail sales, but which are bought near home and except in very small cities are not in the central business district.

Shopping goods include general merchandise, apparel, and furniture, the G.A.F. group amounting to 25.3 percent of all retail sales and are predominantly found in the downtown areas.

All other groups include automobile, lumber, fuel, eating and drinking, and other stores amounting to 38.2 percent of all retail sales. Automobile sales, although at one time in downtown areas, are now generally located on "automobile row" where land values are cheaper and where more space is available for display and garage facilities. Lumber and fuel are usually sold from areas near railroad sidings where land values are cheaper than in the downtown area, and where appearance is not a factor in the esthetic value of the neighborhood. Eating and drinking places, even if downtown, do not add much to parking requirements during the business day since patrons are generally downtown for other reasons such as work or shopping. In the evening parking requirements are normally less and adequate space is then available.

By recognizing that certain retail business establishments are usually grouped in the central business district, it is possible to use 1948 census figures on gross sales and make estimates for problem areas to the extent that the statistics are in common for the area.

Other references are made as to uses made of these same census statistics such as those of

Homer Hoyt in his paper presented at the January 1952 meeting of the Highway Research Board. Sears, Roebuck and Co. has used annual retail sales of $10,000 per car space as a criterion for planning their parking space needs. Supermarkets have used $15,000 per car space, a larger figure than Sears, Roebuck and Co. since the time per sales transaction is not so long.

The relation of urbanized area population and the number of parkers per million dollars of G.A.F. sales is shown graphically in figure 3, while supporting data for each of the 58 cities studied are included in table 5. At first glance it appears that there is a wide range of values obtained, and this is true if no further consideration is given to differences in the economy of individual cities. The nature of dominant employment in cities is an indication of the economy of each city, and statistics are available from census sources classifying employment as to manufacturing, retail business, wholesale business, and service trades.

When the averages are used for each of the conventional census population groups, the average curve clearly indicates that a definite relation exists between parking, G.A.F. sales, and population (figure 3). It should be noted that all of the cities in economic groups I and II with but two exceptions, Monroe and Alexandria, La., are on the average curve or above it, and the curve for these retail cities has been drawn to indicate a reasonable upper limit to be expected for cities within the range of those in these studies.

It appears that cities dominant in industry, groups IV and V, are generally below the average curve, and the industrial curve has been drawn as a reasonable lower limit for cities within the range of those in these studies. Group III cities are generally closer to the average.

Here again it would appear that knowing certain economic characteristics of a city such as population, and dominance in industry or retail trade, it should be possible to estimate parking volumes for a given city by determining the group of cities in which the particular city falls, and assuming that the relation between parking and G.A.F. sales also holds.

Statistical Evaluation

The facts with respect to population, employment, sales, and parking in the discussion on economic groupings of cities on rank-size and on G.A.F. sales seem to indicate that parking volumes bear some relation to the basic economy of urban areas. The third approach in this analysis was to make a statistical evaluation of whatever relation these factors might have, so that some assurance might be placed on estimates of the parking volumes as being within reasonable limits.

Before proceeding with the development of an equation, assurance has to be obtained that a definite relation exists between the particular data, in this case parking volumes, and the independent variables and the extent of this relation measured. By using the multiple regression method,[7] it is possible to measure the degree of reliability of an estimate derived from an equation based upon the interrelation of one series of data (parking volumes) and independent variables or factors.

Factors Studied

Seventeen factors were tested in 31 different combinations. The maximum number of factors used in any combination was four and many combinations were not tested. Several of the factors were closely related with each other such as population, G.A.F. sales, gasoline filling station sales, and the number of employees in retail and wholesale trade. The use of any one of these factors in an equation duplicated to a large extent the use of the others in this group. Factors tested in relation to parking volumes are as follows:

1. 1950 urban area population.
2. Percent of employees in manufacturing.
3. Percent of employees in retail trade.
4. Area of central business district.
5. G.A.F. retail sales.
6. Rural population per square mile.
7. 1950 county registration of vehicles.
8. G.A.F. sales per capita.
9. Gasoline service station sales.
10. Number of employees in retail trade.
11. Number of employees in retail and service trades.
12. Number of employees in retail, wholesale, and service trades.
13. Number of parking spaces.
14. Area of urbanized area.
15. Inbound cordon count of vehicles, 10 a.m.–6 p.m.
16. Curb parking spaces.
17. Percent of all parking spaces at the curb.

A considerable range is represented in the 58 cities in which comparable parking studies have been made. Not only are cities ranging in population from 7,300 to 1,400,000 represented, but different areas of the country are included. Some of the tests were limited to groups of the 58 cities in an effort to stabilize the test within specific groups if it were possible to do so. Groups of cities in which tests were made are as follows:

1. All 58 cities.
2. 27 cities under 50,000 inhabitants.
3. 23 cities of 100,000 inhabitants or more.
4. 13 cities in northeastern U. S.
5. 16 cities in north central U. S.
6. 29 cities in northeast and north central U.S.

Reliability of Estimate

The best adjusted standard error obtained indicated unsatisfactory results (—37 to +59 percent) for all 58 cities regardless of factors used (table 6). Only slightly better results were obtained for the group of cities of 100,000 inhabitants and over (—29 to +41 percent). Best results were obtained when only the 27 cities under 50,000 inhabitants were used.

Trial and elimination of factors indicated that this was the best grouping of cities, and that (1) the cordon count of inbound traffic, 10 a.m. to 6 p.m., (2) total number of parking spaces in the central business district, and (3) number of employees in retail and service trades produced the most satisfactory adjusted standard error (—16 to +19 percent). This means that an equation using these three variables will have about two chances out of three of being within the range of —16 to +19 percent of the parking volume determined from actual field observations.

Factors showing the closest relation to parking volume were found to be:

1. Total number of available parking spaces in the central business district.

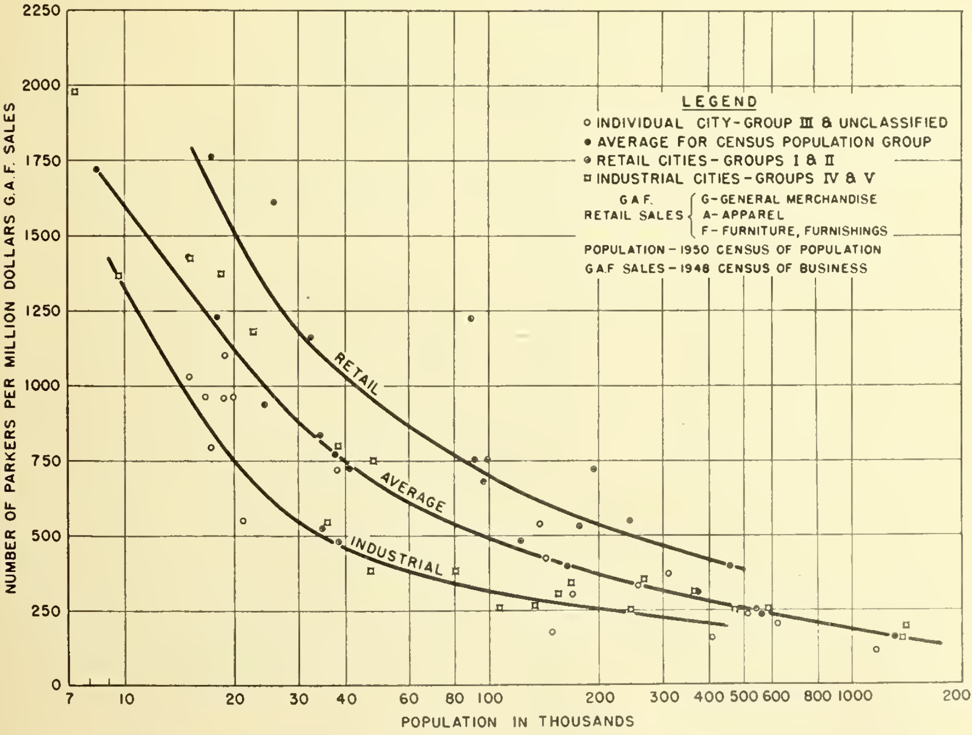

Figure 3.—Relation between urban population and the number of parkers per million dollars of G.A.F. retail sales.

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[7] *Methods of correlation analysis* by Mordecai Ezekiel, John Wiley & Sons, Inc., New York, 1930, chs. 12–13.

2. [illegible] cordon count 10 a.m. to 6 p.m.
3. Number of employees in retail and service trades.
4. G.A.F. sales.
5. [illegible] other service station sales.
6. Urban area population.

Although a significant relation seems to exist between the parking demand and most of the factors tested, as is indicated by the standard error, nevertheless the variation in the original set of data cannot be entirely explained by these factors alone. It is apparent that factors other than those tested also influence parking volumes found in the central business district. Data for these other factors such as use of mass transit facilities to the central business district, proximity to other cities, and volume of parking in neighborhood and suburban shopping centers are not so generally available, cannot be measured in quantitative manner, or cannot be obtained from other sources.

Census information for dollar volume of retail sales and employment are available only for the entire urban area which is not, of course, an exact measurement of activity of the central business district. Then, too, data which have been used do not represent the same time periods. Census years are spread from 1947 for census of manufactures, 1948 for census of business, to 1950 for the census of population, and the parking studies were made between 1945 and 1952. Such disparity in time periods can scarcely be avoided under present conditions, since no other data are available having common base years.

Table 5.—G.A.F. sales and parking volumes in 58 cities in which comparable parking studies have been made

City and State	Urbanized area population	Volume of retail sales: Total	Volume of retail sales: G.A.F.	Volume of retail sales: Percent of total	Number of parkers [1]: Total	Number of parkers [1]: Per million dollars of G.A.F. sales
	Thousands	*Millions*	*Millions*	*Percent*	*Thousands*	
St. Louis, Mo.	1,400.0	$987.7	$357.5	36.2	70.5	197
Cleveland, Ohio	1,383.6	1,178.2	390.1	33.1	64.7	166
Baltimore, Md.	1,161.8	1,052.9	369.7	35.1	40.7	110
Seattle, Wash.	621.5	613.7	212.2	34.6	44.3	209
Providence, R. I.	583.3	328.4	109.6	33.4	28.2	257
Dallas, Texas	538.9	621.9	234.0	37.6	59.2	253
Portland, Oreg.	512.6	574.8	200.2	34.8	47.4	237
Louisville, Ky.	472.7	411.1	130.0	31.6	38.8	298
Miami, Fla.	458.6	388.3	110.9	28.6	44.1	398
Memphis, Tenn.	406.0	461.8	179.9	39.0	28.6	159
Toledo, Ohio	364.3	393.6	118.1	30.0	37.1	314
Omaha, Nebr.	310.3	308.5	90.8	29.4	33.8	372
Syracuse, N. Y.	265.3	290.2	90.2	31.1	26.4	351
Richmond, Va.	258.0	319.5	112.1	35.1	37.8	337
New Haven, Conn.	244.8	206.0	63.5	30.8	15.9	250
Jacksonville, Fla.	242.9	247.8	67.6	27.3	37.0	547
Wichita, Kans.	194.0	201.6	61.5	30.5	44.5	724
Spokane, Wash.	176.0	198.0	60.0	30.3	32.7	545
Harrisburg, Pa.	169.6	140.8	54.0	38.4	16.9	313
Chattanooga, Tenn.	167.8	167.0	50.9	30.5	17.7	348
Reading, Pa.	154.9	140.0	50.8	36.3	15.8	311
Knoxville, Tenn.	148.2	174.8	61.6	35.2	10.7	174
Charlotte, N. C.	140.9	169.7	53.6	31.6	22.6	422
Evansville, Ind.	137.6	139.3	48.1	34.5	25.9	538
Gary, Ind.	133.9	140.4	48.3	34.4	13.0	269
Corpus Christi, Tex.	122.9	123.2	34.5	28.0	16.6	481
Allentown, Pa.	106.8	142.1	55.6	39.1	14.6	263
Lincoln, Nebr.	99.5	118.0	42.9	36.4	32.7	762
Albuquerque, N. Mex.	96.8	114.9	32.2	28.0	21.8	677
Topeka, Kans.	89.1	92.0	28.4	30.9	34.9	1,229
Pawtucket, R. I.	81.4	89.6	26.9	30.0	10.2	379
Lynchburg, Va.	47.7	60.1	18.0	30.0	13.5	750
Anderson, Ind.	46.8	61.3	17.6	28.7	6.7	381
Lake Charles, La.	41.3	44.6	13.4	30.0	9.7	724
Kokomo, Ind.	38.7	44.5	11.9	26.7	9.5	798
Monroe, La.	38.6	62.7	20.1	32.1	9.6	478
Norristown, Pa.	38.1	43.9	14.8	33.7	10.7	723
Easton, Pa.	35.6	58.2	21.4	36.8	11.7	547
Alexandria, La.	34.9	46.8	15.2	32.5	7.9	520
Boise, Idaho	34.4	65.6	20.6	31.4	17.1	830
Reno, Nev.	32.5	71.9	18.5	25.7	21.5	1,162
Roswell, N. Mex.	25.7	32.0	7.7	24.1	12.4	1,610
Walla Walla, Wash.	24.1	41.4	10.6	25.6	10.0	943
Pottstown, Pa.	22.6	33.7	9.0	26.7	10.6	1,178
Uniontown, Pa.	20.5	54.7	19.1	34.9	11.4	597
Anderson, S. C.	19.8	34.6	11.7	33.8	11.3	966
Meadville, Pa.	19.0	33.4	8.9	26.6	9.8	1,101
Portsmouth, N. H.	18.8	23.2	5.0	21.6	4.8	960
Columbus, Ind.	18.4	23.6	5.6	23.7	7.7	1,375
Clovis, N. Mex.	17.3	24.2	5.6	23.1	9.9	1,768
Martinsville, Va.	17.2	21.4	6.7	31.3	5.3	791
Stevens Point, Wis.	16.6	18.4	5.8	31.5	5.6	966
West Chester, Pa.	15.2	29.0	6.3	21.7	6.5	1,032
Huntington, Ind.	15.0	19.7	4.4	22.3	6.3	1,432
Frankfort, Ind.	15.0	19.8	3.7	18.7	5.3	1,445
Wabash, Ind.	10.6	13.1	2.2	16.8	5.8	2,636
Seymour, Ind.	9.6	15.6	3.0	19.2	4.1	1,367
Decatur, Ind.	7.3	10.1	1.4	13.9	2.9	2,071

[1] Average daily number for period 10 a.m.–6 p.m. from parking study reports.

Multiple Correlation Analysis

Three factors, inbound cordon count of vehicles, number of parking spaces, and number of employees in retail and service trades for cities under 50,000 population, were used in a formula most satisfactorily for all groups tested to produce an estimate of probable parking volumes. This estimate in terms of average parking accumulation, which is the average number of vehicles parked at any one time in the central business district, can be expanded to daily parking volumes by the known percentage relation existing between average accumulation and daily parking volume. The formula used for estimating the volume of parking in relation to traffic volumes, parking spaces, and employment for cities of less than 50,000 population is as follows:

$$X = 2547 + 0.0125\ X_2 + 0.0894\ X_3 + 0.0362\ X_4$$

Where:

X = 1000 times the log of the average parking accumulation.[8] (X is a logarithmic number and is converted into the estimated number of parkers by use of the logarithmic tables.)

X_2 = inbound cordon count of vehicles, 10 a.m.–6 p.m.

X_3 = number of parking spaces in the central business district.

X_4 = number of employees in retail and service trades.

The average parking accumulation in percent of the daily number of parkers (10 a.m.–6 p.m.) determined from the analysis of 65 studies, is as follows:

Population group	*Percent*
Under 25,000	12
25,000-50,000	14
50,000-100,000	16
100,000-250,000	18
250,000-500,000	22
500,000-1,000,000	24
1,000,000 and over	30

The equation demonstrates this part of the analysis and the results may be used for comparison with estimates made under the rank-size and G.A.F. sales ratio methods for the smaller cities.

Comparison of Analyses

Each of these three methods appeared to offer possibilities for estimating parking volumes. A comparison of estimates made by each of these three methods was made for 8 cities where comparable parking studies have been made, and where results have been reported since the analysis of the reports from the 58 cities was completed. In 6 of these 8 cities sufficient data have been reported to make an estimate by means of the formula, and in 5 of the 6 cities estimates were within 15 percent of actual observed volumes. An average of the estimates made independently by each of the three methods was within 14 percent of actual observed parking volumes in all of the cities, and in 5 of the 8 cities estimates were

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[8] Adjusted to 1948 by using a straight-line relation based on the population change in a particular city from 1940–50.

Table 6.—Summary of adjusted standard error in percent for each of the groups of factors tested

Test symbol	Factors tested [1]	Solving for—	Number of cities	Adjusted standard error
		ALL CITIES		
A	1, 2, 3	Daily number of parkers per million dollars of G.A.F. sales	53	(2)
Q	1, 5, 9	Daily number of parkers, unadjusted	58	37 to 59
R	1, 5, 9	Average daily parking accumulation, adjusted to 1948	54	42 to 73
		CITIES UNDER 50,000 INHABITANTS		
B	1, 2, 3, 4	Daily number of parkers per million dollars of G.A.F. sales	27	27 to 37
C	1, 4	do	27	26 to 36
D	1	Daily number of parkers, unadjusted	27	24 to 34
E	1, 5	do	27	27 to 37
F	1, 5, 6	do	27	27 to 37
G	1, 5, 7	do	27	27 to 37
I	8, 9, 10	Daily number of parkers, adjusted to 1948	27	24 to 32
J	8, 9, 11	Average daily parking accumulation, adjusted to 1948	27	21 to 26
K	5, 9, 11	do	27	22 to 28
EE	11, 13, 15	do	27	16 to 19
		CITIES OF 100,000 INHABITANTS OR MORE		
L	8, 9, 11	Average daily parking accumulation, adjusted to 1948	23	32 to 47
M	5, 9, 11	do	23	32 to 46
N	4, 9, 11	do	23	29 to 41
O	4, 9, 12	do	23	30 to 42
P	4, 9, 11	Peak parking accumulation, unadjusted	23	29 to 41
		CITIES IN NORTHEASTERN UNITED STATES		
S	4, 9	Average daily parking accumulation, adjusted to 1948	13	23 to 30
T	4, 9, 11	do	13	24 to 31
U	13	do	13	19 to 24
V	9, 13	do	13	20 to 25
W	13, 14	do	13	17 to 21
X	11, 13, 14	do	13	14 to 16
CC	15, 16	do	13	22 to 29
		CITIES IN NORTH CENTRAL UNITED STATES		
Y	11, 13, 14	Average daily parking accumulation, adjusted to 1948	16	34 to 52
Z	13, 15	do	16	30 to 42
AA	13, 15, 17	do	16	30 to 43
BB	15, 16	do	16	21 to 27
		CITIES IN NORTHEAST AND NORTH CENTRAL UNITED STATES		
DD	15, 16	Average daily parking accumulation, adjusted to 1948	29	22 to 28
		CITIES BELOW THE AVERAGE CURVE [3]		
H	1, 5	Daily number of parkers, unadjusted	17	35 to 55

[1] For a list of factors tested, refer to items numbered 1-17 on page 255.
[2] Adjusted standard error is 366 parkers per million dollars of G.A.F. sales.
[3] Refer to figure 3, page 255.

Table 7.—Comparison of estimates and actual observed number of parkings

City and State	Year of study	Population	Estimated number of parkers from—				Observed number of parkers	Average estimate in percent of total
			Rank size	G.A.F. sales	Formula	Average		
		Thousands						*Percent*
Ogden, Utah	1952	57.1	18,100	15,800	24,100	19,300	18,548	104
Lexington, Ky.	1952	55.5	18,100	22,100	(1)	20,100	21,641	93
Norristown, Pa.	1949	38.2	10,500	11,100	11,850	11,150	10,660	105
Steubenville, Ohio	1952	35.7	10,500	10.200	(1)	10,350	11,979	87
Bristol, Va.–Tenn.	1950	32.7	9,900	10,500	9,300	9.900	8,864	112
Fond du Lac, Wis.	1950	29.9	9,800	8,960	9.430	9,400	10,893	86
Butler, Pa.	1951	23.5	9,650	11,985	12,417	11,350	11,812	96
Coatesville, Pa.	1951	13.8	5,200	5,650	6,742	5,860	6,246	94

[1] Insufficient data received.

within 7 percent of actual observed volumes (table 7).

The following example is illustrative of the methods used in making estimates using the city of Bristol as an example. The twin city of Bristol is located on the Virginia-Tennessee State line, has essentially one business district, and a combined population of 33,000 persons. It is located on the Norfolk and Western Railroad, and U. S. Highway No. 11 passes through, connecting eastern Tennessee with the northeastern States. It is on the edge of the Great Smoky National Park, an area of increasing recreational importance, and on the edge of eastern Tennessee which has grown industrially with the power developments of the Tennessee Valley Authority. Considerable diversification in manufacturing for a city of this size is reported. The proximity of Kingsport and Johnson City within a 25-mile distance to the south and west tends to reduce the normal trading area in these directions.

Bristol has a retail trade-manufacturing employment ratio of 1:2.0 which is typical of a group IV city (see figure 1 and table 2). Employment per 100 population of 31 percent is also typical of a group IV city, but the percentage increase in population of 38 percent is more nearly typical of a group II city. The percentage of employment in service trades of 7.5 percent is also typical of group IV cities, so it seems reasonable to consider Bristol as being included in this group.

Bristol ranks between 40th and 41st with respect to population, and between 38th and 39th with respect to G.A.F. sales. With 5,644 persons employed in manufacturing, it would rank between 35th and 36th for this factor which is about five places higher than population, a pattern similar to the rank-size patterns of the cities in group IV. This confirms the assumption that Bristol appears to be similar to a group IV city. In this group parking has a ranked position of about one position lower than population, and thus Bristol would be expected to have a rank in parking of 41.5 (40.5+1). This would mean that in figure 2 an estimated 9,900 daily parkers could normally be expected.

For the second type of estimate based on G.A.F. sales data, modest dominance in manufacturing together with proximity to Kingsport and Johnson City suggests lower than average retail sales. If we assume 675 parkers per million dollars of G.A.F. sales (fig. 3) which is about half way between the average city and a dominant industrial city, and knowing that Bristol has an annual G.A.F. sales volume of $15.5 million, it is estimated that there would be 10,500 daily parkers in Bristol.

By formula the estimate is computed to be 9,300 parkers. The actual study showed that 8,864 parkers were observed. The formula estimate was less than 5 percent greater than the observed volume, and the average of all three estimates was within 12 percent of the actual volume. Average estimates for some of the other cities were closer to observed volumes, and formula estimates were not uniformly the closest (table 7).

Summary

The findings of this study have not been presented with the idea that a definite and unqualified basis for making estimates of parking volumes is now available. They are presented rather to indicate that it should be possible to make a relatively quick appraisal of probable parking requirements for the central business district as a [illegible] origins and destinations of parkers are needed for the planning of specific highway or parking facility improvements, then more detailed information must be obtained by other means.

The three types of analyses indicate that some basic relations seem to exist between parking volumes and the basic economy of the community, and that the average of the estimates of parking made by the various methods can be used with reasonable assurance of reliability. Before any extensive use can be made of the estimates for specific locations, however, studies should be made to demonstrate how the estimates can be used in relation to parking habits and current land and building usage. A further conclusion may be inferred. If these relations exist between the volume of parking in the central business district and the size of a city, and the amount of retail trade and certain kinds of employment, then similar relations must exist with respect to travel since parking is directly related to travel in that vehicles are parked at the end of each trip. The application of such relations with respect to urban travel appears to be of much greater significance because entire urban areas must be considered which involves the movement of vehicles rather than their parking.

Standard Plans for Highway Bridge Superstructures: a new publication

The Bureau of Public Roads has just published *Standard Plans for Highway Bridge Superstructures*, in which are presented standard plans providing complete details for various types of superstructures for highway bridges. The publication is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $1.00 a copy.

Standard Plans for Highway Bridge Superstructures is intended to serve as a useful guide to State, county, and local highway departments in developing designs for bridges of adequate strength and economical proportions on primary, secondary, and urban highways. The plans should be particularly valuable to the smaller highway departments with limited engineering staffs.

Included in the several series of drawings are detailed plans for I-beams, riveted deck plate girders, welded deck plate girders, reinforced concrete slabs, reinforced concrete T-beams, reinforced concrete box girders, post-tensioned precast reinforced concrete deck girders, and precast reinforced concrete deck girders. One series of I-beam spans involves 3-span continuous units. All other types of spans are simply supported. One series of simply supported I-beam spans is designed for composite action. The design of the reinforced concrete slab spans has been based on participating curbs.

Designs are included for two widths of roadway, 24 feet and 28 feet. The span lengths for the simply supported structures range from a minimum of 20 feet to a maximum of 140 feet. The 3-span continuous units vary from 130 feet to 260 feet in overall length, with the end spans having a length 80 percent of the center span. The ranges in span lengths are based on the economy and suitability of the different types of superstructures. The live load capacity for all types of superstructures is H15-44 for 24-foot roadway widths and H20-S16-44 for 28-foot roadway widths, with the exception of the precast reinforced concrete deck girder spans which are designed for a loading of H15-S12-44.

All of the designs are in accordance with the "Standard Specifications for Highway Bridges," adopted by the American Association of State Highway Officials in 1953.

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

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Public Roads Administration Annual Reports:
1943; 1944; 1945; 1946; 1947. (*Free from Bureau of Public Roads*)

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UNIFORM VEHICLE CODE

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act (1945). 15 cents.

Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act. 15 cents. (revised 1952)

Act III.—Uniform Motor-Vehicle Civil Liability Act (1944). 10 cents.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act. 15 cents. (revised 1952)

Act V.—Uniform Act Regulating Traffic on Highways. 20 cents. (revised 1952)

Model Traffic Ordinance. 20 cents. (revised 1952)

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.

United States System of Numbered Highways together with the Federal-Aid Highway System (also shows in color National forests, parks, and other reservations). 5 by 7 feet (in 2 sheets), scale 1 inch equals 37 miles. $1.25.

United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

MISCELLANEOUS PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.

Construction of Private Driveways, No. 272MP (1937). 10 cents.

Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.

Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.

Federal Legislation and Regulations Relating to Highway Construction (1948). *Out of print.*

Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.

Highway Accidents (1938). 10 cents.

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Highway Bridge Location, No. 1486D (1927). 15 cents.

Highway Capacity Manual (1950). 65 cents.

Highway Needs of the National Defense, House Document No. 249 (1949). 75 cents.

Highway Practice in the United States of America (1949). 75 cents.

Highway Statistics (*annual*):
1945, 35 cents. 1948, 65 cents.
1946, 50 cents. 1949, 55 cents.
1947, 45 cents. 1951, 60 cents.

Highway Statistics, Summary to 1945. 40 cents.

Highways in the United States, *nontechnical* (1951). 15 cents.

Highways of History (1939). 25 cents.

Identification of Rock Types (1950). 10 cents.

Interregional Highways, House Document No. 379 (1944). 75 cents.

Legal Aspects of Controlling Highway Access (1945). 15 cents.

Local Rural Road Problem (1950). 20 cents.

Manual on Uniform Traffic Control Devices for Streets and Highways (1948). 75 cents.

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.

Public Control of Highway Access and Roadside Development (1947). 35 cents.

Public Land Acquisition for Highway Purposes (1943). 10 cents.

Results of Physical Tests of Road-Building Aggregate (1953). $1.00

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Selected Bibliography on Highway Finance (1951). 55 cents.

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Taxation of Motor Vehicles in 1932. 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.

Transition Curves for Highways (1940). $1.50.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).

Bibliography on Highway Lighting (1937).

Bibliography on Highway Safety (1938).

Bibliography on Land Acquisition for Public Roads (1947).

Bibliography on Roadside Control (1949).

Express Highways in the United States: a Bibliography (1945).

Indexes to PUBLIC ROADS, volumes 17–19 and 23.

Title Sheets for PUBLIC ROADS, volumes 24, 25, and 26.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1953

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	[illegible]	$12,162	$5,799	194.4	$13,176	$6,578	147.6	$39,051	$20,041	462.0	$64,389	$32,418	804.0
Arizona	1,112	2,190	1,497	55.9	1,070	758	7.4	7,093	5,029	108.2	10,353	7,284	171.5
Arkansas	1,663	13,085	6,804	372.5	6,219	3,114	149.6	9,488	4,706	223.7	28,792	14,624	745.8
California	1,710	5,550	2,969	40.5	11,892	4,159	43.5	97,843	49,321	269.8	115,285	56,449	353.8
Colorado	2,482	3,991	2,258	101.8	3,244	1,835	47.9	16,809	9,084	184.3	24,044	13,177	334.0
Connecticut	6,804	1,870	1,041	3.6	477	237	.6	9,289	4,558	28.7	11,636	5,836	32.9
Delaware	3,120	605	305	.8				4,260	2.123	20.3	4,865	2,428	21.1
Florida	4,583	14,270	7,286	210.1	7,087	3,826	132.7	17,036	8,598	236.4	38,393	19,710	579.2
Georgia	4,521	16,923	8,656	383.6	7,036	3,510	134.1	34,167	16,083	434.4	58,126	28,249	952.1
Idaho	2,400	7,195	4,497	112.4	2,781	1,767	46.0	11,786	7,442	207.2	21,762	13,706	365.6
Illinois	[illegible]	26,721	14,543	130.7	13,755	6,927	31.3	65,147	34,324	495.5	105,623	55,794	657.5
Indiana	10,629	33,269	17,900	149.2	6,677	3,351	30.9	23,850	12,150	103.0	63,796	33,401	283.1
Iowa	2,580	13,906	7,486	331.1	4,188	3,115	75.4	18,308	9,202	885.9	36,402	19,803	1,292.4
Kansas	3,513	9,383	4,694	814.1	3,925	1,995	315.7	16,155	7,747	755.1	29,463	14,436	1,884.9
Kentucky	3,012	8,997	4,777	106.6	7,297	3,648	106.8	21,208	11,167	246.9	37,502	19,592	460.3
Louisiana	1,729	20,185	10,122	133.5	3,549	1,772	17.5	26,357	12,745	142.7	50,091	24,639	293.7
Maine	973	5,433	2,851	37.0	2,279	1,119	17.4	13,324	6,508	103.9	21,036	10,478	158.3
Maryland	8,261	5,391	2,869	50.3	4,595	2,232	19.2	7,006	3,892	34.9	16,992	8,993	104.4
Massachusetts	4,124	8,643	4,416	18.5	6,961	3,358	3.9	41,821	19,951	34.9	57,425	27,725	57.3
Michigan	1,318	28,868	14,967	366.1	11,170	5,594	138.9	49,808	21,277	289.5	89,846	41,838	794.5
Minnesota	4,414	9,239	5,083	811.9	2,509	1,283	130.5	17,207	8,953	432.6	28,955	15,319	1,375.0
Mississippi	1,098	12,645	6,438	378.4	4,534	2,146	113.7	20,755	10,590	567.4	37,934	19,174	1,059.5
Missouri	6,937	14,810	7,497	762.6	4,636	2,480	111.9	56,903	28,049	461.7	76,409	38,026	1,336.2
Montana	4,532	12,472	7,609	309.0	4,543	2,916	82.2	13,104	7,843	200.8	30,119	18,368	592.0
Nebraska	7,742	22,881	11,898	757.6	2,183	1,303	70.2	10,577	5,965	236.0	35,641	19,166	1,063.8
Nevada	3,030	5,485	4,591	91.4	725	607	26.9	4,276	3,542	74.3	10,486	8,740	192.6
New Hampshire	3,938	1,900	950	7.9	990	494	7.6	3,655	1,950	16.7	6,545	3,394	32.2
New Jersey	2,637	5,904	2,927	58.0	8,188	3,889	8.5	25,527	12,124	19.9	39,619	18,940	86.4
New Mexico	1,140	1,164	747	30.6	2,960	1,863	77.5	8,990	5,664	177.7	13,114	8,274	285.8
New York	13,903	60,620	32,030	101.4	53,814	26,708	38.1	157,758	72,736	507.1	272,192	131,474	646.6
North Carolina	3,233	21,351	10,419	388.6	3,893	1,820	55.7	29,705	13,904	388.4	54,949	26,143	832.7
North Dakota	2,552	3,552	1,784	662.8	1,087	544	185.2	7,789	4,047	494.5	12,428	6,375	1,342.5
Ohio	4,763	17,329	7,971	106.6	12,249	5,931	42.8	86,822	41,698	123.7	116,400	55,600	273.1
Oklahoma	6,517	12,661	7,159	187.1	5,621	2,962	80.5	14,151	7,514	169.4	32,433	17,635	437.0
Oregon	1,695	1,602	956	30.8	817	465	9.8	12,634	7,640	188.2	15,053	9,061	228.8
Pennsylvania	2,749	13,425	5,478	4.1	42,549	20,357	64.0	82,882	41,325	240.0	138,856	67,160	308.1
Rhode Island	1,948	2,227	1,113	26.5	1,133	566	5.3	9,831	4,912	22.2	13,191	6,591	54.0
South Carolina	2,741	9,392	5,237	203.0	3,309	1,622	204.6	14,459	7,260	243.4	27,160	14,119	651.0
South Dakota	854	6,354	3,793	380.6	2,026	1,092	99.4	8,581	4,983	469.2	16,961	9,868	949.2
Tennessee	3,848	11,152	5,554	329.2	7,657	4,459	189.3	30,621	13,392	312.1	49,430	23,405	830.6
Texas	8,366	6,416	3,213	114.0	9,543	5,142	319.3	60,292	33,044	810.0	76,251	41,399	1,243.3
Utah	633	2,860	2,191	44.7	708	532	8.1	10,562	8,057	159.4	14,130	10,780	212.2
Vermont	657	3,315	1,777	38.6	269	135	.2	8,294	4,181	45.5	11,878	6,093	84.3
Virginia	499	11,112	5,049	150.3	7,041	3,107	94.1	29,738	14,347	194.6	47,891	22,503	439.0
Washington	1,274	9,160	4,804	116.4	1,643	860	43.1	16,254	8,553	113.8	27,057	14,217	273.3
West Virginia	[illegible]	7,554	3,804	40.4	4,730	2,386	3.9	13,447	6,733	98.9	25,731	12,923	143.2
Wisconsin	3,503	7,209	4,027	122.5	5,972	2,960	79.5	25,386	12,816	271.1	38,567	19,803	473.1
Wyoming	382	1,537	992	23.1	1,928	990	29.4	7,510	4,864	165.6	10,975	6,846	218.1
Hawaii	1,249	3,221	1,580	7.8	41	21		10,818	5,176	13.9	14,080	6,777	21.7
District of Columbia	876	7,538	3,529	6.0	3,071	1,338	.1	10,069	4,730	3.1	20,678	9,597	9.2
Puerto Rico	3,638	10,540	4,839	51.2	304	147	2.0	13,532	6,498	38.7	24,376	11,484	91.9
TOTAL	[illegible]	555,264	290,776	9,955.8	318,051	160,020	3,649.8	1,351,995	679,038	12,527.2	2,225,310	1,129,834	26,132.8

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The printing of this publication has been approved by the Director of the Bureau of the Budget, January 5, 1952.

Contents of this publication may be reprinted. Mention of source is requested.

Evaluation of Air-Entraining Admixtures for Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by FRANK H. JACKSON,[1] Chief, Non-Bituminous Section, and
ALBERT G. TIMMS, Supervising Highway Physical Research Engineer

This investigation was undertaken for the purpose of determining the acceptability of a number of commercial admixtures which were available at the time for use in the production of air-entrained portland cement concrete for highway pavements and bridges. The complete program involved the evaluation of 27 materials, each of which was tested in concrete to determine its ability to entrain air, as well as its effect on flexural and compressive strength and on durability. As the result of the evaluation study, 26 of the 27 materials which were received for test have been classified as acceptable for use as air-entraining admixtures.

PRIOR to 1950, information regarding the effectiveness of most of the materials available commercially for use in the production of air-entrained portland cement concrete, other than that furnished by the manufacturers themselves, was exceedingly meager and, as a result, the State highway departments and other users were reluctant to specify them. In fact, the only materials which were generally accepted prior to 1950 were the four which had been initially accepted by Committee C-1 of the American Society for Testing Materials under its standard procedure for evaluating additions for use in the manufacture of air-entraining portland cement.[2]

The term *addition* is generally understood to cover materials, other than gypsum, which are interground with portland cement clinker during the process of manufacturing the cement, while the term *admixture* is generally understood to cover those materials which are added to the concrete either prior to or during the mixing operation.

Although the A.S.T.M. acceptance procedure covered the use of the four materials only as additions, engineers in general have assumed that materials which are satisfactory for use in making air-entraining cement will also be satisfactory for use as admixtures in producing air-entrained concrete. Therefore, inasmuch as all four of these products were also being marketed as admixtures, they came to be generally accepted for this purpose.

The rapidly increasing use of air-entrained concrete during the past decade, and the consequently increasing demand for materials which would entrain air, has resulted in the promotion of many new products, all claimed to be equal to or superior in air-entraining properties to the four which had been previously accepted. Very few data upon which to appraise these materials as air-entraining admixtures were available, however. It was to supply such data that the evaluation program described in this article was undertaken.[3]

Classification of Materials

As an initial step, a questionnaire was sent to all manufacturers of air-entraining admixtures known to the Bureau, together with a request for samples of their products. The purpose of the questionnaire was to obtain data regarding the nature of the materials, results of prior tests, extent of use, and field performance. This request resulted in the submission of 27 samples representing the products of 19 manufacturers. The materials have been classified with respect to their major constituents into seven groups, as follows:

Group A.—Salts of wood resins.
Group B.—Synthetic detergents.
Group C.—Salts of sulfonated lignin.
Group D.—Salts of petroleum acids.
Group E.—Salts of proteinaceous materials.
Group F.—Fatty and resinous acids and their salts.
Group G.—Organic salts of sulfonated hydrocarbons.

Of the 27 materials submitted, seven were classified in group A, seven in group B, four in group C, two in group D, two in group E, four in group F, and one in group G. Seventeen of the samples were in liquid form, five were powders, four were in flake form, and one was a semisolid.

The classification of the individual materials into groups, the physical state in which each was received, and a brief statement of the probable source of each material and its essential chemical composition, are shown in table 1.

With the exception of the semisolid, which had to be neutralized, all of the admixtures had been neutralized prior to submission for test. They were added to the concrete in the physical state in which received except Nos. 7, 12, 13, 14, and 15, which were dissolved in water before use in order to insure uniform distribution in the mix.

Basis of Acceptance and Summary of Results

The criteria for acceptance which had been established by the Bureau provides that, when the admixture is used in an amount sufficient to entrain between 3 and 6 percent air in concrete of the proportions normally used for pavements and bridges:

1. The flexural and compressive strength of the concrete at the ages of 3 days, 28 days, and 1 year shall be not less than 88 percent of the strength of similar concrete of the same cement content and consistency but without the admixture, and

2. The durability of the concrete as measured by its resistance to alternate freezing and thawing in water, using a slow cycle of 24 hours, shall be substantially improved as compared to the durability of similar concrete of the same cement content and consistency but without the admixture.

As the result of the evaluation program, the Bureau has officially approved for use on Government contracts in the National Parks and National Forests, 26 of the 27 materials which were under consideration, including the 4 which previously had been accepted by the A.S.T.M. (see p. 278). One material met all of the required tests, but approval was deferred because the material, as submitted by the manufacturer, was not in suitable form for use as an admixture. Except for a few relatively minor deviations, the 27 admixtures met the 88-percent strength requirement at the three ages of test. They also complied with the requirement for durability, although in this respect the various materials differed considerably in performance. These differences could not be related directly to total air content, but it is possible that they were influenced by differences in the distribution or spacing of the air voids. This possibility is a subject for further study.

A detailed description of the test procedures employed in this work and a further discussion of the results follows.

Scope of Work and Materials Used

Three series of tests were conducted: Series I, a study of the effect of each of the 27 admixtures on the compressive and flexural strength of concrete, using a well-graded crushed limestone, 1½-inch maximum size, as coarse aggregate; Series II, a similar study using a siliceous gravel of identical size and grading as in Series I but covering only a selected group of 15 of the 27 admixtures; and Series III, a study of the effect

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[1] Mr. Jackson retired on October 1, 1953.

[2] See A.S.T.M. tentative specification C 175-48T. The acceptance procedure outlined in this specification was withdrawn in 1950. The materials which had been accepted prior to withdrawal were known commercially as neutralized Vinsol resin, Darex AEA, N-Tair, and Airalon.

[3] A companion article in this issue covers the chemical analysis of the same air-entraining admixtures that are considered here. The samples are identically numbered in both cases. In this article the samples are grouped according to source, but in the other study they are grouped according to treatment for chemical analysis.

Table 1.—Classification of admixtures

No.	Physical state	Probable source	Chemical constituents identified
		GROUP A: SALTS OF WOOD RESINS	
1	Liquid	Pine wood stumps	Sodium soaps of wood resins.
2	do	do	do.
3	do	do	do.
4	do	do	do.
5	do	do	do.
6	do	do	Sodium and potassium soaps of wood resins.
7	Powder	do	Sodium soaps of wood resins.
		GROUP B: SYNTHETIC DETERGENTS	
12	Flake	Synthesized from petroleum fractions	Alkyl aryl sulfonate and sodium sulfate.
13	do	do	do.
14	do	do	do.
15	do	do	do.
16	Liquid	do	Alkyl aryl sulfonate.
17	do	do	Alkyl aryl sulfonate and sodium sulfate.
26	do	Direct source uncertain [1]	Alkyl aryl sulfonate and proteinaceous material.
		GROUP C: SALTS OF SULFONATED LIGNIN	
18	Powder	Byproduct of paper pulp industry [2]	Calcium lignosulfonate.[3]
20	do	do [2]	Calcium lignosulfonate and calcium chloride.[3]
21	do	do [2]	do.[3]
22	do	do [2]	do.[3]
		GROUP D: SALTS OF PETROLEUM ACIDS	
11	Liquid	Petroleum refining processes	Sodium soaps of carboxylic acids (most likely naphthenic).
27	do	do	Sodium soaps of petroleum acids (possibly naphthenic) dissolved in petroleum oil.
		GROUP E: SALTS OF PROTEINACEOUS MATERIALS	
24	Liquid	Animal hide industry	Calcium salts of complex proteinaceous acids.
25	do	do	do.
		GROUP F: FATTY AND RESINOUS ACIDS AND THEIR SALTS	
8	Liquid	Byproduct of paper pulp industry [4]	Sodium soaps of lignin, rosin, and fatty acids.
9	do	do [4]	Triethanolamine and sodium soaps of lignin, rosin, and perhaps fatty acids.
10	do	do [4]	Ammonium and sodium soaps of lignin and rosin along with appreciable unsaponified matter.
28	Semisolid	Byproduct of vegetable oil processing	Fatty acids and unsaponified matter.
		GROUP G: ORGANIC SALTS OF SULFONATED HYDROCARBONS	
23	Liquid	Byproduct of petroleum refining processes	Triethanolamine salts of sulfonic acids.

[1] Analysis shows two major components present, separately derived from the animal hide industry and the petroleum industry.

[2] Acid process. [3] Stated to contain also a small quantity of alkyl aryl sulfonate. [4] Alkaline process.

of each of the 27 admixtures on durability, as measured by resistance to alternate freezing and thawing. In Series III the same coarse aggregate was used as in Series I, except that the maximum size was 1 inch. This maximum aggregate size was due to the smaller size specimen (3 by 4 by 16-inch) used in the freezing tests as compared with 6 by 6 by 21-inch beams and 6 by 12-inch cylinders used for strength tests.

Mix Proportions

The mix data for all series are given in tables 2 and 2A. For each series, two proportions were used, one for the base or reference mix without admixture and the other for the mixes containing entrained air. All mixes were designed to have a cement content of 6.0±0.2 sacks of cement per cubic yard, with sufficient water to produce a slump of approximately 3.0 inches. For each series, the actual weight of coarse aggregate per sack of cement was the same for both types of concrete. In the case of the air-entrained mixes, however, the quantity of sand was reduced in order to compensate for the volume of added air, thus maintaining a constant cement content. The reduction amounted to 20 pounds per sack of cement in the case of Series I, 17 pounds in the case of Series II, and 15 pounds in the case of Series III. The actual weights of aggregates per sack of cement are shown in table 2A.

In the case of Series I, the quantities of admixture used, air contents, and slumps are the average of three rounds of tests made on different days. For Series II, the corresponding values are the average of two rounds. In the case of Series III, the individual specimens were not cast separately but were sawed from slabs 16 by 24 by 4 inches in depth which were cast from single batches of concrete. The values shown for Series III in table 2 are, therefore, the results of tests on the single batches from which the slabs were cast.

The procedure for determining the proper quantity of admixture to use in each case was as follows:

For the first round of Series I, the first batch contained either the amount of admixture recommended by the manufacturer or the minimum of the recommended range, if so shown. If the resulting air content was within the 3- to 6-percent range set up as the criteria for acceptance, the batch was used. If the air content was within the range but close to either limit, the quantity of admixture for the second and third rounds was adjusted either up or down as needed to bring the air content closer to the middle of the permissible range. In the case of admixture 6, for example, 0.50 ounce of admixture per sack (the minimum of the recommended range) was used in round one. The resulting air content was 3.1 percent. Therefore, for the second and third rounds the quantity was increased to 0.63 ounce per sack, resulting in an air content of 3.8 percent for round two and 4.2 percent for round three. The average values for the three rounds were, therefore, 0.59 ounce of admixture per sack and 3.7 percent air content.

If the observed air content for the first batch was outside the limits of 3 to 6 percent, the batch was discarded and a new batch using an adjusted quantity of admixture was made. In the three cases where no recommendations were made by the manufacturers, trial batches were run to establish the correct quantities.

For Series II, the quantities of the admixtures were, in general, raised slightly as compared to the quantities used in Series I. This adjustment was made because of the lower sand content of the gravel mixes, experience having shown that without such correction the air contents in Series II would have been substantially lower than in Series I.

For Series III, it was decided to use the optimum quantity as determined in Series I. It was recognized that, because of the higher sand content of the mixes in Series III, the use of the same quantity of admixture would probably increase the air content. However, it was felt that some increase in air content would be justified in this mix due to the smaller maximum coarse aggregate size. As will be noted from table 2, the resulting air contents show considerable variation. They will be discussed in a later section of this report.

Fabrication of Specimens

A total of 10 mixing days was required to complete fabrication of the specimens for Series I. For each of the 27 admixtures, three specimens of a kind were cast on each of 3 mixing days—one for test at 3 days, one for test at 28 days, and one for test at 1 year. For the base mix, three specimens of a kind were cast on each of the 10 mixing days required to complete the program—one for test at 3 days, one for test at 28 days, and one for test at 1 year. In the case

Table 2.—Concrete mix data [1]

Admixture			Series I: Limestone coarse aggregate				Series II: Gravel coarse aggregate				Series III: Limestone coarse aggregate			
No.	Physical state	Amount recommended	Admixture	Water	Air	Slump	Admixture	Water	Air	Slump	Admixture	Water	Air	Slump
BASE MIX: WITHOUT AIR-ENTRAINING ADMIXTURE														
		Oz./sack	*Oz./sack*	*Gal./sack*	*Pct.*	*In.*	*Oz./sack*	*Gal./sack*	*Pct.*	*In.*	*Oz./sack*	*Gal./sack*	*Pct.*	*In.*
None	----	----	----	5.6	1.0	3.1	----	5.0	1.0	3.0	----	6.1	1.5	2.6
GROUP A: SALTS OF WOOD RESINS														
1	Liquid	1–1½	1.00	5.2	5.1	3.3	1.02	4.6	4.2	3.0	1.00	5.6	6.0	2.7
2	do	1	.66	5.1	5.0	3.0	----	----	----	----	.67	5.6	6.6	3.0
3	do	½	.43	5.2	5.1	3.2	----	----	----	----	.48	5.6	6.8	3.0
4	do	½–1	.75	5.1	4.1	3.0	.92	4.7	4.4	3.1	.75	5.6	5.3	2.8
5	do	1.043	1.04	5.1	4.1	3.2	----	----	----	----	1.04	5.6	6.2	3.7
6	do	½–1	.59	5.1	3.7	3.2	.79	4.6	5.1	3.0	.63	5.4	4.8	2.7
7	Powder	.24	.12	5.1	5.1	3.6	.12	4.7	4.6	3.0	.13	5.6	7.7	3.5
GROUP B: SYNTHETIC DETERGENTS														
12	Flake	0.2–0.3	0.14	5.1	5.1	3.3	0.14	4.7	4.6	3.5	0.16	5.6	6.8	3.0
13	do	.08–.15	.08	5.1	5.1	3.5	----	----	----	----	.03	5.5	4.6	3.0
14	do	1	.16	5.1	5.2	3.0	----	----	----	----	.19	5.4	8.5	3.5
15	do	----	.06	5.1	5.2	3.0	----	----	----	----	.05	5.4	6.1	2.8
16	Liquid	1	.14	5.1	4.7	3.2	.19	4.6	4.9	3.6	.17	5.4	7.5	3.2
17	do	----	.14	5.1	4.9	3.0	.16	4.6	4.5	3.1	.13	5.4	6.3	3.4
26	do	⅕	.18	5.1	5.4	3.6	.19	4.6	4.4	3.1	.17	5.5	7.0	3.6
GROUP C: SALTS OF SULFONATED LIGNIN														
18	Powder	4.00	4.00	4.8	5.4	3.6	----	----	----	----	4.00	4.9	6.6	2.4
20	do	6.40	6.40	4.8	5.3	3.4	----	----	----	----	6.40	4.8	6.1	2.0
21	do	8.00	8.00	4.8	5.0	3.1	----	----	----	----	8.00	4.9	7.3	3.0
22	do	16.00	16.00	4.8	4.3	3.0	----	----	----	----	16.00	5.0	3.6	1.8
GROUP D: SALTS OF PETROLEUM ACIDS														
11	Liquid	¼–½	0.34	5.1	3.5	2.9	0.54	4.7	4.4	3.2	0.41	5.6	5.0	3.0
27	do	0.80	.80	5.1	3.9	3.2	----	----	----	----	.81	5.6	4.7	2.5
GROUP E: SALTS OF PROTEINACEOUS MATERIALS														
24	Liquid	[2] ¼–½	1.74	5.0	4.6	3.8	2.27	4.6	4.3	3.0	1.68	5.5	5.8	3.5
25	do	4	4.00	5.0	4.5	3.6	4.71	4.6	3.8	3.7	4.00	5.4	5.5	2.6
GROUP F: FATTY AND RESINOUS ACIDS AND THEIR SALTS														
8	Liquid	¾–1	0.75	5.2	3.9	2.9	1.37	4.7	4.5	3.5	0.76	5.6	3.5	2.4
9	do	½	.50	5.1	3.9	2.8	.67	4.7	4.5	3.4	.50	5.6	4.7	2.7
10	do	½–1	.58	5.1	3.8	2.9	1.03	4.6	4.0	3.5	.63	5.6	4.5	2.2
28	Semisolid	----	.37	5.0	4.5	3.2	----	----	----	----	----	5.6	7.7	3.6
GROUP G: ORGANIC SALTS OF SULFONATED HYDROCARBONS														
23	Liquid	¾–1	0.75	5.1	3.9	2.9	0.87	4.7	3.8	3.2	0.76	5.6	4.8	2.4

[1] See also table 2A. [2] Percent by weight of cement.

of all of the air-entrained concretes the values reported are, therefore, the averages of 3 tests made on different days, whereas the values reported for the base mix are the average of 10 tests made on different days.

A total of 4 mixing days was required to complete the specimens for Series II. For each of the 15 selected admixtures, three specimens of a kind were cast on each of 2 mixing days. From the concrete made the first day, two specimens of a kind were made for test at 28 days and one for test at 1 year. From the concrete made the second day, one specimen of a kind was cast for test at 28 days and two for test at 1 year. For the base mix, three specimens of a kind were made on each of the four mixing days. The specimens made during the first two days were tested at 28 days and those made the last two days at 1 year.

For each of the 27 admixtures and the base mix used in Series III, one 16 by 24 by 4-inch slab was cast. Two mixing days were required for this portion of the program, one base mix slab being cast on each mixing day. After a preliminary curing period of 14 days in moist air followed by 35 days in laboratory air, each slab was prepared for test by sawing it into seven 3 by 4 by 16-inch beams.

Table 2A.—Proportions of concrete mixes by oven-dry weight, in pounds

Series	Coarse aggregate	Base mix			Air-entrained mix		
		Cement	Aggregate		Cement	Aggregate	
			Fine	Coarse		Fine	Coarse
I	Limestone	94	195	350	94	175	350
II	Gravel	94	170	370	94	153	370
III	Limestone	94	215	315	94	200	315

All mixing was done in an open-pan type Lancaster mixer of 1¾ cubic-foot capacity. The following mixing cycle was employed: The cement plus the damp sand was first mixed for 30 seconds, after which the estimated total quantity of mixing water plus the admixture, if any, was added and the mortar mixed for a further period of 1 minute. The coarse aggregate in a saturated, surface-dry condition was then added and the concrete mixed for an additional 2 minutes, making a total mixing period of 3½ minutes. Air contents were determined by the pressure method, A.S.T.M. test method C 231-49T. Consistency was controlled by means of the standard slump test.

Table 3.—Series I: Flexural and compressive strength of concrete with limestone coarse aggregate

Admixture	Air content [1]	Modulus of rupture [2] at—						Compressive strength [3] at—					
		3 days		28 days		365 days		3 days		28 days		365 days	
		Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio
BASE MIX: WITHOUT AIR-ENTRAINING ADMIXTURE													
	Pct.	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*
None	1.0	455	100	790	100	800	100	1,760	100	4,230	100	5,530	100
GROUP A: SALTS OF WOOD RESINS													
1	5.1	430	97	730	92	730	91	1,750	99	3,950	93	4,980	90
2	5.0	420	94	770	97	810	101	1,540	87	3,910	92	5,110	92
3	5.1	455	102	710	90	710	89	1,680	95	3,950	93	5,030	91
4	4.1	400	90	745	94	790	99	1,730	98	3,880	92	5,390	97
5	4.1	455	102	775	98	800	100	1,810	103	4,130	98	5,590	101
6	3.7	475	107	820	104	810	101	2,110	120	4,390	104	6,070	110
7	5.1	420	94	730	92	740	92	1,660	94	3,950	93	5,100	92
GROUP B: SYNTHETIC DETERGENTS													
12	5.1	370	83	740	94	730	91	1,630	93	3,920	93	4,700	85
13	5.1	390	88	750	95	745	93	1,580	90	3,870	91	4,910	89
14	5.2	425	96	745	94	770	96	1,710	97	4,110	97	5,180	94
15	5.2	435	98	780	99	725	91	1,900	108	4,350	103	5,300	96
16	4.7	425	96	765	97	725	91	1,870	106	4,170	99	5,310	96
17	4.9	455	102	750	95	740	92	1,920	109	4,410	104	5,420	98
26	5.4	400	90	745	94	800	100	1,800	102	4,090	97	5,190	94
GROUP C: SALTS OF SULFONATED LIGNIN													
18	5.4	535	120	800	101	800	100	2,380	135	4,950	117	6,260	113
20	5.3	575	129	830	105	840	105	2,430	138	4,880	115	6,210	112
21	5.0	560	126	820	104	880	110	2,730	155	5,100	121	6,450	117
22	4.3	635	143	845	107	850	106	3,130	178	5,300	125	6,800	123
GROUP D: SALTS OF PETROLEUM ACIDS													
11	3.5	440	99	760	96	770	96	1,810	103	3,950	93	5,460	99
27	3.9	455	102	745	94	790	99	1,980	112	4,000	95	5,440	98
GROUP E: SALTS OF PROTEINACEOUS MATERIALS													
24	4.6	455	102	780	99	780	98	1,900	108	4,300	102	5,760	104
25	4.5	470	106	835	106	795	99	2,010	114	4,220	100	5,620	102
GROUP F: FATTY AND RESINOUS ACIDS AND THEIR SALTS													
8	3.9	440	99	770	97	770	96	1,830	104	4,130	98	5,180	94
9	3.9	480	108	840	106	815	102	1,870	106	4,450	105	5,070	92
10	3.8	510	115	855	108	870	109	2,070	118	4,330	102	6,000	108
28	4.5	455	102	770	97	760	95	1,930	110	4,370	103	5,530	100
GROUP G: ORGANIC SALTS OF SULFONATED HYDROCARBONS													
23	3.9	470	106	775	98	795	99	1,920	109	4,240	100	5,420	98

[1] Air content determined by A.S.T.M. tentative method C 231-49T.

[2] Specimens were 6 by 6 by 21-inch beams tested in accordance with A.S.T.M. standard method C 78-49. Each value for the base mix is the average of 10 tests and for all other groups is the average of 3 tests. Ratio values for relative strength are based on the strengths for the base mix.

[3] Specimens were 6 by 12-inch cylinders tested in accordance with A.S.T.M. standard method C 39-49. Each value for the base mix is the average of 10 tests and for all other groups is the average of 3 tests. Ratio values for relative strength are based on the strengths for the base mix.

Testing Procedures

All strength specimens were molded, cured, and tested strictly in accordance with applicable A.S.T.M. procedures. All specimens were continuously moist cured until tested. Of the seven beams which were sawed from each slab cast in connection with the Series III program, two were subjected to freezing and thawing, two were tested for flexural strength at the start of the freezing test, and three were held for further tests. After sawing, the specimens for the freezing and thawing test and the corresponding control specimens were stored in laboratory air for 10 days and then immersed in water for 7 days prior to test. The specimens held for future tests were stored in moist air.

A 24-hour cycle was used in making the freezing and thawing tests. Specimens were frozen in water at —10° F. for 18 hours and thawed in water at 70° F. for 6 hours. Each specimen was frozen in an individual metal container of such size as to provide about one-eighth-inch clearance on all sides. The specimens in the containers were placed vertically in the freezing chamber. The refrigerant, in this case alcohol, entirely surrounded the containers to a height of about 18 inches. After each cycle, the specimens were removed from the containers and turned upside down. The water in the containers was also changed.

Resistance to the effects of alternate freezing and thawing was determined periodically during the test by measuring changes in the dynamic modulus of elasticity of the concrete, with equipment of the type generally used for this purpose. Inasmuch as, for a given specimen, changes in dynamic E are proportional to changes in the square of the natural frequency of vibration (N^2) it is possible to use the reduction in N^2 directly as a measure of deterioration. This procedure avoids the necessity of calculating values for dynamic E and is the one customarily used. Freezing and thawing was continued until the specimens showed a loss in N^2 of approximately 40 percent, or for 200 cycles, whichever occurred first. They were then broken in flexure.

In order to express in a single figure the loss in N^2 as related to the number of cycles required to produce that loss, a value known as the durability factor *DF* is used. The values for *DF* shown in table 5 were calculated as follows:

$$DF = (100-L)\left(\frac{n}{200}\right)$$

Where:

DF = durability factor.

L = loss in N^2 at n cycles (from table 5).

n = number of cycles at which the loss in N^2 reaches 40 percent, or 200 if loss in N^2 does not reach 40 percent by the end of the test (200 cycles).

The strength ratios reported in table 5 are the ratios of the flexural strengths of specimens after freezing and thawing expressed as percentages of the strengths of the corresponding control specimens, tested at the time the freezing and thawing test was started.

Discussion of Test Results

The results of these tests are shown in tables 2–5 and in figures 1–4. The data will be discussed in terms of the three principal objectives of the investigation. These were to determine, for each admixture, (1) its ability to entrain air in concrete, (2) its effect on the compressive and flexural strength of concrete, and (3) its effect on the durability of concrete.

Ability to Entrain Air

Table 2 shows, for each admixture and for each series, the amount of admixture to be used, as recommended by the manufacturer, the amount actually used, and the corresponding air and water content and slump of the concrete. It will be noted that, in most cases, the amount of admixture which was used was either the same as, or reasonably close to, the amount recommended. Principal exceptions were admixtures 14 and 16, where only about one-seventh of the recommended amount was used, and admixture 7, where only about half of the recommended quantity was used. The tendency, in general, was to use somewhat less than the amounts recommended.

In Series I and II (the strength tests) the amount of air entrained in the concrete was in every case within the usually specified limits of 3 to 6 percent. Actually the total range for the 27 admixtures was less than 2 percent (3.5 to 5.4 percent) with only 9 values out of 42 falling below 4.0 percent. The materials in group A, the salts of wood resins, in group B, the synthetic detergents, and in group C, the salts of sulfonated lignin, gave in general somewhat higher air contents than the materials in groups D, E, F, and G. This, of course, was on the basis of using an amount of admixture either the same as or close to the amount recommended. Increasing the quantity above the recommended amount would, of course, have raised the air content. However, as previously explained, adjustments of this sort were not made so long as the air content obtained with the recommended amount of admixture fell well within 3 to 6 percent.

As already noted, it was decided to use the same quantities of admixture in Series III (the durability tests) as were used in Series I, even though it was realized that, because of the higher percentage of sand, higher air contents would probably be obtained. From table 2 it will be seen that the air contents in Series III were, in general, not only considerably higher than in Series I and II but that they were also more variable. The total range was from 3.5 to 8.5 percent, with 14 values above 6.0 percent, 5 values of 7.0 percent or more, and 2 values below 4.0 percent. Two possibilities occur as to the reasons for these comparatively wide variations. In the first place, each value in Series III was the result of only one test on one batch of concrete (the batch required for casting the 16 by 24 by 4-inch slab), whereas in Series I and II each value was the average of one test on each of three or two batches made on different days. In the second place, the higher sand mix used in Series III was probably somewhat more critical with respect to variables which affect air content than the low sanded mixes used in Series I and II. However, as will be explained later, there appeared to be little or no relation between durability and total air content provided entrained air was present, so that from the standpoint of evaluating the individual admixtures these comparatively wide variations in total air content were unimportant.

It may also be noted from table 2 that the consistency of the concrete was quite uniform throughout the tests. In Series I and II the maximum range in slump in 42 tests was only 1 inch, the maximum value being 3.8 inches and the minimum value 2.8 inches. In the case of Series III the range was somewhat greater, being 1.9 inches, with the maximum 3.7 inches and the minimum 1.8 inches. It is probable that these variations in slump accounted at least in part for the variations in air content obtained in Series III.

For each series the water contents of the various air-entrained concretes were about the same, with the exception of the four admixtures in group C, which required, on the average, about 0.4 gallon of water per sack less than the admixtures in the other groups. With this exception, the reduction in water content due to air entrainment was about 0.5 gallon per sack with a tendency towards a somewhat greater reduction in the case of Series III (high sand content, 1-inch maximum aggregate size) as compared to Series I (low sand content, 1½-inch maximum aggregate size).

Table 4.—Series II: Flexural and compressive strength of concrete with gravel coarse aggregate

Admixture	Air content [1]	Modulus of rupture [2] at—				Compressive strength [3] at—			
		28 days		365 days		28 days		365 days	
		Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio
BASE MIX: WITHOUT AIR-ENTRAINING ADMIXTURE									
	Pct.	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*
None	1.0	675	100	650	100	4,200	100	5,080	100
GROUP A: SALTS OF WOOD RESINS									
1	4.2	695	103	610	94	4,050	96	4,760	94
4	4.4	690	102	620	95	3,880	92	4,860	96
6	5.1	700	104	600	92	3,670	87	4,990	98
7	4.6	635	94	575	88	3,910	93	4,750	94
GROUP B: SYNTHETIC DETERGENTS									
12	4.6	675	100	630	97	4,010	95	4,620	91
16	4.9	630	93	545	84	3,780	90	4,220	83
17	4.5	655	97	525	81	3,840	91	4,720	93
26	4.4	675	100	590	91	3,780	90	4,770	94
GROUP D: SALTS OF PETROLEUM ACIDS									
11	4.4	640	95	630	97	3,750	89	4,730	93
GROUP E: SALTS OF PROTEINACEOUS MATERIALS									
24	4.3	680	101	550	85	4,020	96	5,120	101
25	3.8	695	103	570	88	4,100	98	4,900	96
GROUP F: FATTY AND RESINOUS ACIDS AND THEIR SALTS									
8	4.5	680	101	575	88	3,760	90	4,570	90
9	4.5	695	103	575	88	3,770	90	4,660	92
10	4.0	720	107	630	97	3,650	87	5,010	99
GROUP G: ORGANIC SALTS OF SULFONATED HYDROCARBONS									
23	3.8	665	99	595	92	3,870	92	4,950	97

[1] Air content determined by A.S.T.M. tentative method C 231-49T.

[2] Specimens were 6 by 6 by 21-inch beams tested in accordance with A.S.T.M. standard method C 78-49. Each value for the base mix is the average of six tests and for all other groups is the average of three tests. Ratio values for relative strength are based on the strengths for the base mix.

[3] Specimens were 6 by 12-inch cylinders tested in accordance with A.S.T.M. standard method C 39-49. Each value for the base mix is the average of six tests and for all other groups is the average of three tests. Ratio values for relative strength are based on the strengths for the base mix.

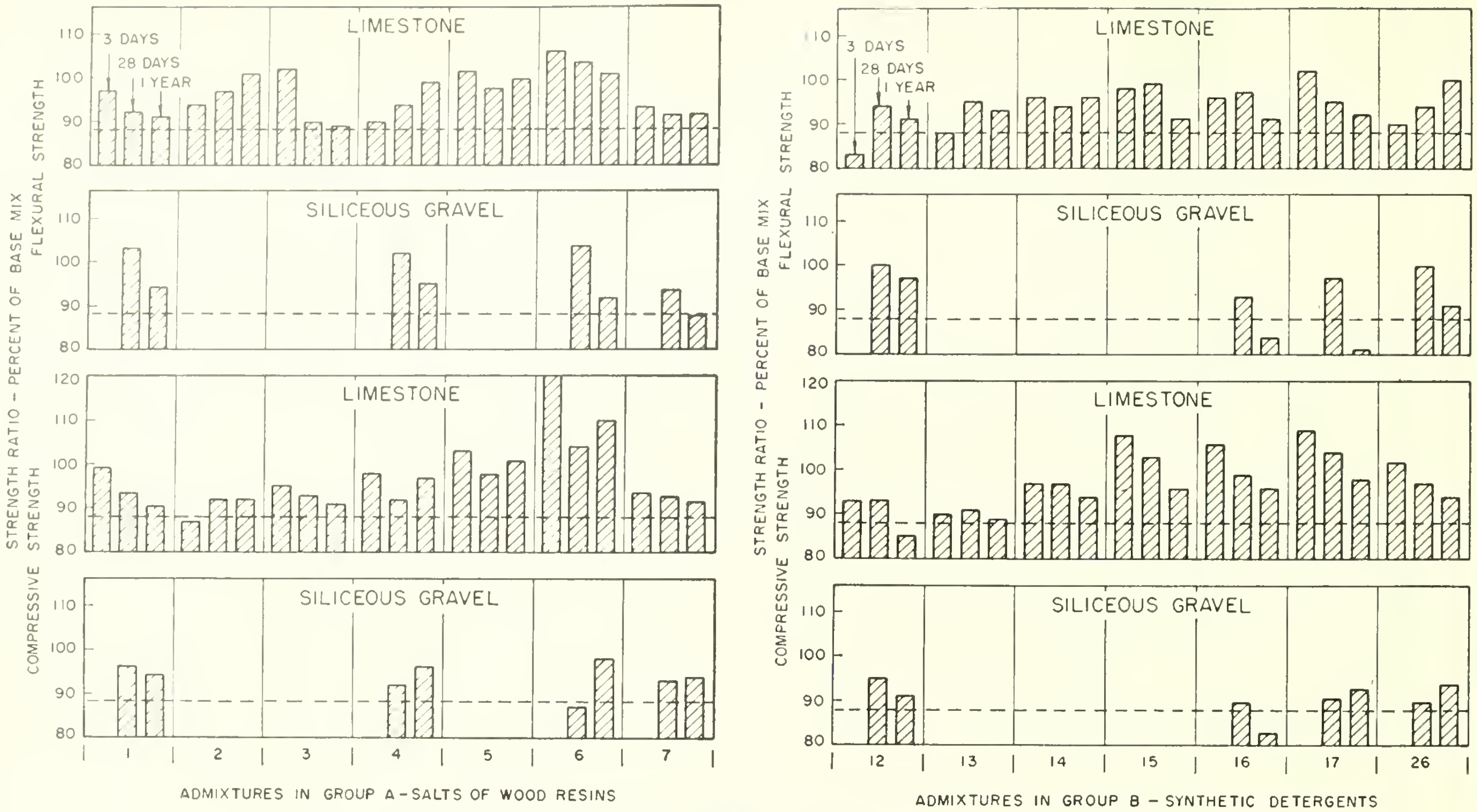

Figure 1.—Effect of air-entraining admixtures on strength of concrete: Groups A and B.

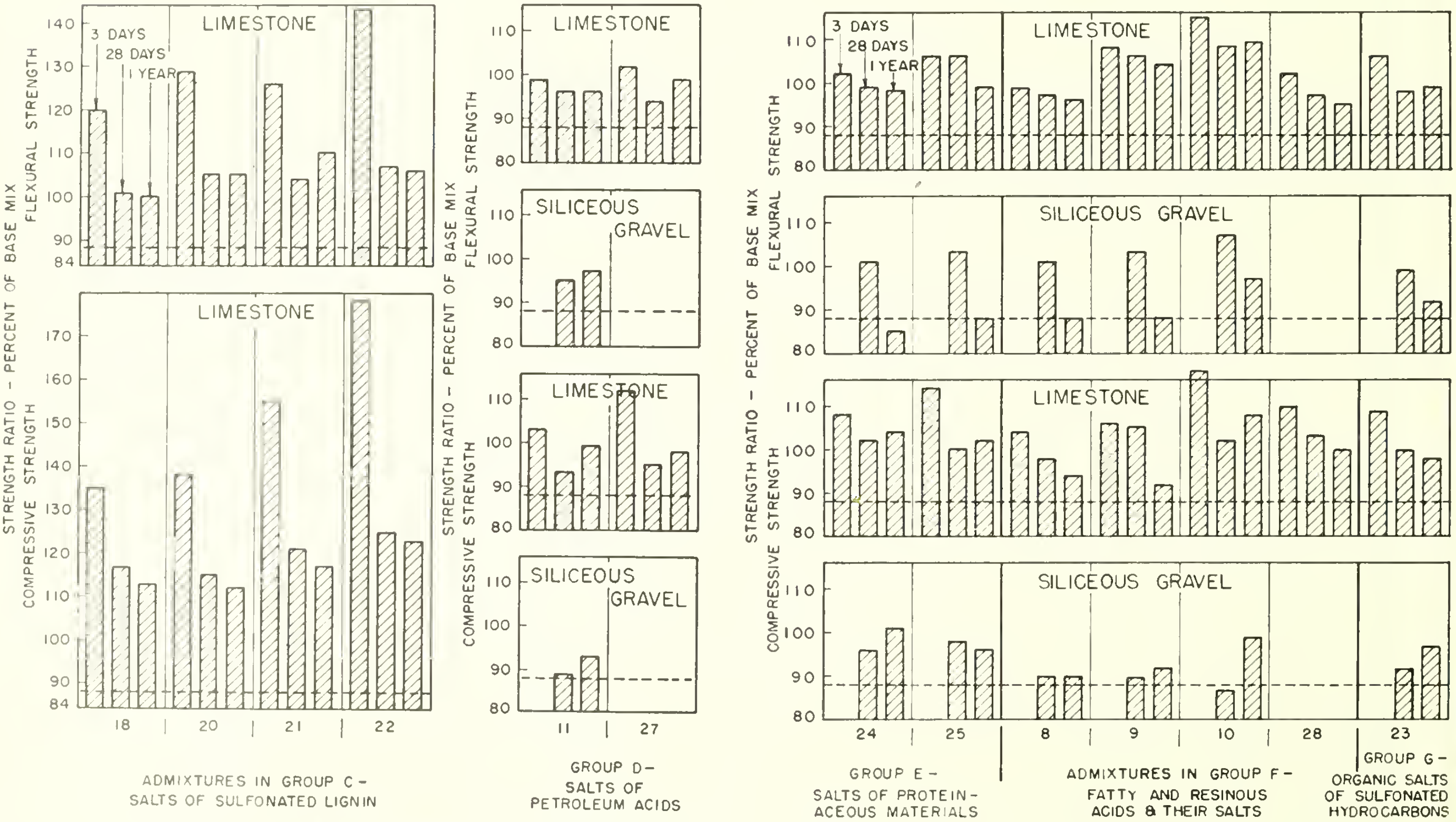

Figure 2.—Effect of air-entraining admixtures on strength of concrete: Groups C–G.

Effect on Strength

The flexural and compressive strengths of the crushed limestone concrete containing each of the 27 admixtures and the base mix (Series I) are shown in table 3, with the corresponding data for the gravel concrete for 15 admixtures only (Series II) in table 4. These tables also show, for each type of test and for each age tested, the ratios of the strengths developed with the admixtures, expressed as percentages of the strengths of the corresponding concretes without admixture. The strength ratios for both series are also plotted graphically in figures 1 and 2. It will be convenient, in discussing the trends as shown by these graphs, to consider the seven groups of admixtures in order.

Group A

The strength ratios for the seven admixtures in group A (the salts of wood resins) are shown in the left half of figure 1. It is at once apparent that in practically all cases the strengths, both in flexure and compression, are at least 88 percent of the strengths of the corresponding base mix. In fact, out of a total of 58 values there are only 2 which are less than 88 percent—admixture 2, for which the compressive strength ratio of limestone concrete at 3 days was 87 percent, and admixture 6, for which the compressive strength ratio of gravel concrete at 28 days was 87 percent. It will be seen that failure to comply with the 88-percent requirement does not occur more than once with any admixture. On the basis of these data all seven of the materials in group A have been accepted as substantially meeting the requirement for strength.

There seems also to be a tendency for the flexural strength ratios to decrease with age. This tendency is quite definite in the case of the gravel concrete and appears in some degree in the limestone concrete in five out of the seven cases, although two of the admixtures, Nos. 2 and 4, show a distinct trend in the opposite direction. These trends are not so well defined in the case of compressive strength. Even here, however, there is a tendency for the relative strength of the limestone concrete to decrease with age.

In general, the strength level of the concretes containing the neutralized wood resin admixtures is about what would be expected with materials of this type in every case except for admixture 6. Although all of these materials were supposed to be salts of wood resins with nothing added, the comparatively high strengths (particularly at 3 days) developed by the concrete containing admixture 6 would indicate the possibility that an accelerator or activating agent of some sort was present in this case.

Group B

The corresponding strength ratios for the seven admixtures in group B (the synthetic detergents) are shown in the right half of figure 1. Again, most of the values are 88 percent or more of the base mix. In this group, however, we find several which are substantially below 88 percent, the lowest being 81 percent. In two cases (admixtures 12 and 16), two out of the ten values show ratios of less than 88 percent whereas, in the case of admixture 17, one out of ten values is less than 88 percent.

As in group A, there is a tendency in group B for the strength ratios to decrease with age. Of the 22 comparisons in the right half of figure 1, 17 show this tendency in some degree. A definite reversal is indicated in only three instances. These are admixture 26, limestone concrete in flexure and gravel concrete in compression, and admixture 17, gravel concrete in compression. On the basis of the data, all seven of the admixtures in group B have been accepted as substantially meeting the requirement for strength.

Group C

The strength ratios developed by the crushed limestone concrete containing the four admixtures in group C (salts of sulfonated lignin) are shown in the left side of figure 2. All of these materials (the products of one manufacturer) developed substantially higher strengths, particularly at 3 days, than any of the other admixtures. In the case of three of the four admixtures in this group, Nos. 20, 21, and 22, high early strength was probably due, at least in part, to the presence of substantial amounts of calcium chloride in the admixture. In fact, there seems to be a fairly good relation between the amount of calcium chloride in the admixtures, as revealed by chemical analysis, and the 3-day strength. However, the fact that strength ratios as high as 135 in compression and 120 in flexure at 3 days were developed by admixture 18 which, according to the manufacturer's statement, contained no calcium chloride, would indicate that some other factor, possibly the dispersing effect of the calcium lignosulfonate on the cement, contributed to the high strengths which were obtained. It is also interesting to note that, in the case of all four of the group C materials, concrete of the required consistency was obtained with water contents approximately 0.3 to 0.4 gallon per sack of cement less than was required for the other materials (see table 2).

Group D

Strength ratios for the two materials in group D (salts of petroleum acids) are shown in the center of figure 2. All of the values are above the 88-percent limit set for evaluation purposes. How-

Table 5.—Series III: Summary of freezing and thawing tests

Admixture	Air content	Loss in N^2 at— cycles						Cycles at 40 percent loss	Durability factor	Modulus of rupture		
		10	50	100	150	180	200			Control specimen	Frozen and thawed specimen [1]	Str'gth ratio
BASE MIX: WITHOUT AIR-ENTRAINING ADMIXTURE												
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*			*P.s.i.*	*P.s.i.*	*Pct.*
None	1.5	16	[2] 54	-----	-----	-----	-----	27	8	890	280 (32)	31
GROUP A: SALTS OF WOOD RESINS												
1	6.0	4	8	24	35	40	-----	180	54	770	375 (180)	49
2	6.6	5	8	22	34	39	-----	182	55	715	355 (180)	50
3	6.8	5	8	24	34	41	-----	178	53	690	360 (180)	52
4	5.3	5	9	28	39	52	-----	155	46	860	360 (180)	42
5	6.2	4	7	17	28	34	[3] 41	[3] 195	[3] 59	800	-----	-----
6	4.8	2	3	15	24	28	34	200+	66	840	420 (200)	50
7	7.7	4	6	21	35	42	-----	174	52	725	360 (180)	50
GROUP B: SYNTHETIC DETERGENTS												
12	6.8	3	7	21	34	41	-----	178	53	690	360 (180)	52
13	4.6	5	6	20	30	33	37	200+	63	770	455 (200)	59
14	8.5	5	6	21	32	38	-----	186	56	800	400 (180)	50
15	6.1	5	7	17	27	29	35	200+	65	735	425 (200)	59
16	7.5	2	5	17	26	30	36	200+	64	795	390 (200)	49
17	6.3	5	6	18	28	33	40	200	60	780	440 (200)	56
26	7.0	5	6	18	28	32	39	200+	61	800	345 (200)	43
GROUP C: SALTS OF SULFONATED LIGNIN												
18	6.6	4	5	14	22	26	31	200+	69	970	445 (200)	46
20	6.1	4	8	17	25	28	34	200+	66	920	495 (200)	54
21	7.3	2	6	17	26	29	35	200+	65	950	385 (200)	41
22	3.6	2	11	24	37	44	-----	163	49	1000	490 (180)	49
GROUP D: SALTS OF PETROLEUM ACIDS												
11	5.0	5	7	22	36	49	-----	159	48	915	340 (180)	37
27	4.7	5	10	26	38	48	-----	156	47	730	305 (180)	42
GROUP E: SALTS OF PROTEINACEOUS ACIDS												
24	5.8	3	7	22	32	37	44	189	57	860	410 (200)	48
25	5.5	2	7	18	32	40	-----	180	54	935	425 (180)	45
GROUP F: FATTY AND RESINOUS ACIDS AND THEIR SALTS												
8	3.5	8	22	[4] 46	-----	-----	-----	80	24	790	280 (85)	35
9	4.7	4	11	33	[5] 43	-----	-----	118	35	780	380 (125)	49
10	4.5	3	5	21	28	36	55	185	56	920	320 (200)	35
28	7.7	6	7	23	31	34	39	200+	61	650	370 (200)	57
GROUP G: ORGANIC SALTS OF SULFONATED HYDROCARBONS												
23	4.8	5	13	28	39	46	-----	152	46	760	350 (180)	46

Each value is the average of two tests.

[1] Figures in parentheses indicate number of cycles at which test was made.

[2] Tested at 32 cycles.

[3] Specimens damaged after 180 cycles. Value estimated from probable curve.

[4] Tested at 85 cycles.

[5] Tested at 125 cycles.

ever, there is the same tendency for the ratios to decrease with age as has been previously noted, except that in the case of these materials a general tendency to recover strength somewhat between 28 days and 1 year is also noted. These two admixtures therefore met the requirement for strength.

Groups E, F, and G

Strength ratios for the materials in group E (salts of proteinaceous materials), group F (fatty and resinous acids and their salts), and group G (organic salts of sulfonated hydrocarbons) are shown in the right side of figure 2. It will be noted that, in the case of the limestone concrete, all of these materials meet the 88-percent strength requirement, both in flexure and compression. In the case of the gravel concrete in flexure, however, one of the materials (No. 24) is below the 88-percent limit at 1 year, while in compression one material (No. 10) is slightly under at 28 days. Again, the tendency for the strength ratios to decrease with age is apparent, although the differences in some instances are not significant. These materials have all been accepted as complying substantially with the requirement for strength.

Effect on Durability

The data from the freezing and thawing tests are shown in tabular form in table 5. The results are presented as the loss in N^2 at 10, 50, 100, 150, 180, and 200 cycles, the number of cycles required to cause a 40-percent reduction in N^2, the durability factor, the modulus of rupture determined both at the conclusion of the freezing and thawing test and on the corresponding control specimens, and the ratio of the flexural strength of the frozen and thawed specimen expressed as a percentage of the strength of the corresponding control specimen. The reductions in N^2 plotted against the number of cycles of freezing and thawing are shown for each admixture in figure 3, while in figure 4 the durability factors and the corresponding ratios of the strengths of the frozen and thawed specimens to the strengths of the control specimens have been plotted in bar diagram form.

In view of the fact that the durability factor is generally considered to be a satisfactory measure of the effect of alternate freezing and thawing, this value will be used as the primary basis of comparison of the various admixtures. This will be followed by a brief discussion of the value of the strength test for the same purpose.

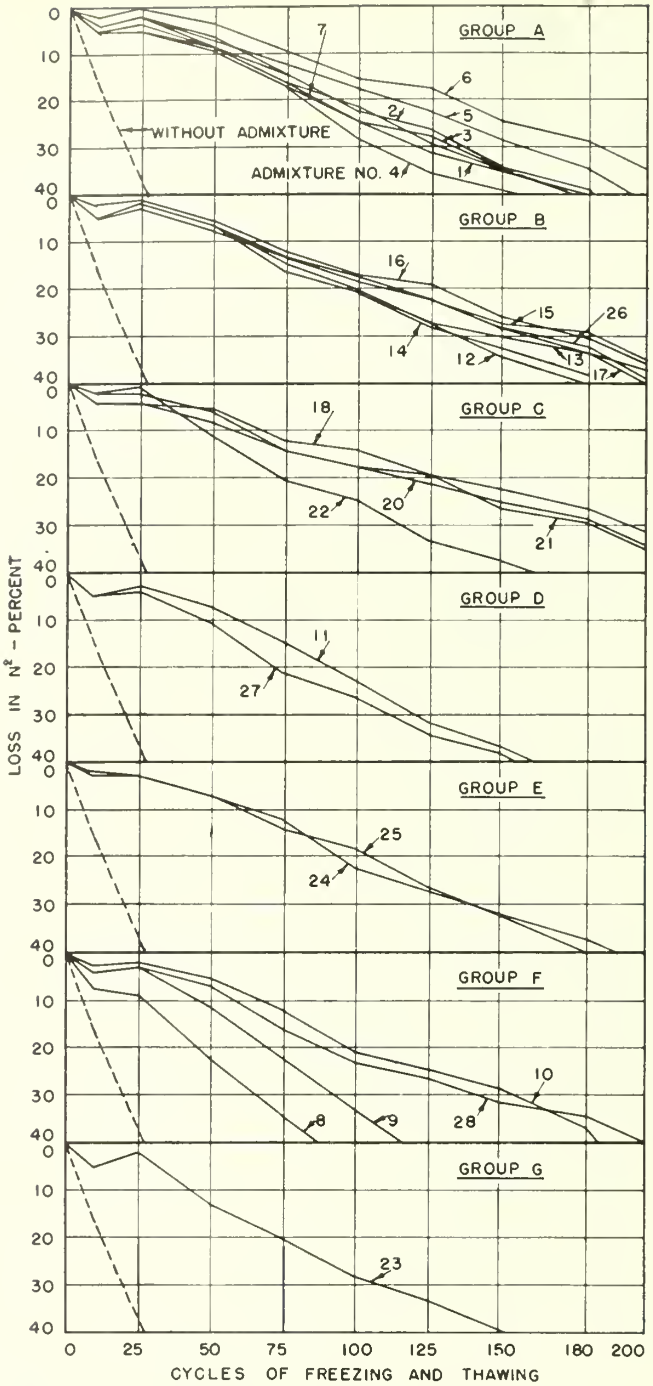

Figure 3.—Resistance of concrete to alternate freezing and thawing as influenced by different air-entraining admixtures.

Durability Factor Comparisons

For the seven materials in group A (salts of wood resins), it will be noted that the durability factors range from a low of 46 to a high of 66, as compared with 8 for the base mix without air entrainment. Air contents varied from 4.8 to 7.7 percent. However, there appears to be little or no relation between total air content and durability. For example, the admixture with the lowest air content, No. 6, shows the highest durability factor, 66, whereas the admixture with the highest air content, No. 7, has next to the lowest durability factor. These apparent discrepancies may be related to a variable which, although not as yet clearly understood, is being intensively studied—the effect on durability of size and distribution of the air voids in the concrete. This is a variable which, so far, has been impossible to control in the laboratory. There is good reason to believe that variations in the size and spacing of the bubbles of entrained air in concrete may well cause marked differences in durability even though the total air content remains the same, and it is quite possible that uncontrolled variations of this kind may account for the lack of correlation between total air content and durability factor shown by these tests.

In group B (the synthetic detergents), the durability factors vary from a low of 53 to a high of 65. Five of the seven values are 60 or more, indicating a considerably higher general level for this group than for group A. Air contents vary from 4.6 to 8.5. As in group A, however, there is little correlation between total air content and durability, indicating in this case also the influence of some uncontrolled factor.

The materials in group C (salts of sulfonated lignin), with the single exception of admixture 22, attained a higher general level of durability than those of any of the other groups. In this case there does seem to be some relation between air content and durability, at least so far as admixture 22 is concerned. This particular mem-

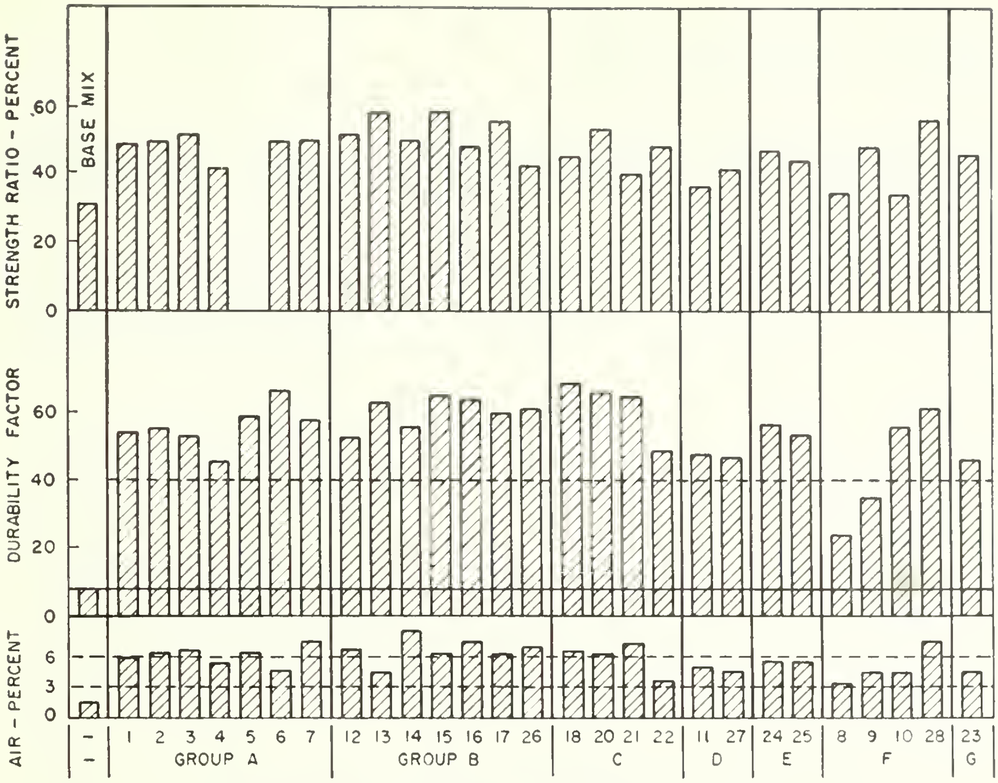

Figure 4.—Results of freezing and thawing tests.

ber of the group had an air content of 3.6 percent, the lowest of the group by a wide margin. It also showed the lowest durability factor of any of the four materials in this group. Although it may be without significance, it is interesting to note that a chemical analysis of admixture 18, having the highest durability factor in the group, revealed the presence of only a trace of calcium chloride. All of the other materials in the group contained an appreciable quantity of this salt.

The durability factors of the two materials in group D (salts of petroleum acids) were somewhat lower than the general level for groups A, B, and C. Durability factors of two of the materials in group F (fatty and resinous acids and their salts)—admixtures 8 and 9—were also comparatively low. The other two materials in group F, as well as the two admixtures in group E (salts of proteinaceous materials), showed durability factors at about the same general level as the first two groups but somewhat lower than group C. As in other instances, the durability factors of the materials in these groups bear little relation to total air content except possibly for admixture 8. The concrete containing this admixture, with air content of only 3.5 percent, showed the least durability factor of any of the materials tested.

The one admixture classified in group G (organic salts of sulfonated hydrocarbons) produced concrete with a durability factor of 46, somewhat below the general level of groups A, B, and C. This concrete contained 4.8 percent air.

Flexural Strength Loss

The last column of table 5 shows the relative effect of freezing and thawing on flexural strength. In making comparisons between admixtures on the basis of strength loss, the number of cycles at which the test was made must be taken into consideration. For example, the strength ratio of 31 shown for the base mix cannot be compared directly with the corresponding ratios for any of the air-entrained concretes because the base concrete was tested after only 32 cycles whereas for most of the admixtures the strength tests were made after either 180 or 200 cycles. It was to take care of this variable that the durability factor was developed. For any given number of cycles, however, the strength ratios can be compared directly. For example, in the case of the 12 concretes which were tested for strength at 180 cycles, the two methods with one exception correlate reasonably well. Except for this case (admixture 11), the range in durability factor was from 46 to 55, with the corresponding range in strength ratio from 42 to 52.

Summary of Durability Tests

Summarizing, it has been shown that all of the admixtures included in this program, when used in amounts sufficient to entrain 3 to 6 percent air, substantially improved the durability of concrete. There was, however, a considerable range in the relative resistance of the various air-entrained concretes, the durability factors varying from a low of 24 for admixture 8 to a high of 69 for admixture 18. It has been shown that these variations are not related to the total air content of the concrete but that they may very well be due to some as yet uncontrolled variable related to air content, such as the size and spacing of the air bubbles. On the basis of these tests it is concluded that all of the admixtures included in this study, except No. 28 which, as has been stated, was submitted in such form as to require further processing before use, should be classified as satisfactory provided they are used in quantities which will entrain between 3 and 6 percent air in the concrete. A list of the acceptable admixtures appears on page 278.

NEW PUBLICATIONS

Now available is the ***Annual Report of the Bureau of Public Roads, Fiscal Year 1953***, which may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at 25 cents a copy. The 45 pages of text and 27 pages of statistical tables describe the operation of the Federal-aid highway program during the fiscal year ended June 30, 1953. Also reported are the Bureau's construction, research, and foreign activities during the period.

Also available is the Bureau's ***Highway Statistics, 1952***, the eighth of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of highways, and highway mileage. The 142-page publication may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at 75 cents a copy. The full series of the annual bulletins are available from the Superintendent of Documents, as indicated on the inside back cover of PUBLIC ROADS.

Chemical Analysis and Sources of Air-Entraining Admixtures for Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by WOODROW J. HALSTEAD
and BERNARD CHAIKEN, Chemists

This article presents the results of an investigation of the practicability of applying chemical analyses as a means of identifying and controlling the uniformity of the various air-entraining admixtures used in concrete. Some such means of identification and control is needed, since the general practice at the present time is to approve a specific material for use after extended laboratory tests or actual experimental installations have shown it to be satisfactory. Subsequent purchases of the same material are then approved either without further tests or on the basis of a limited number of short-term physical tests, but such limited tests do not necessarily give the consumer any assurance that the material purchased under the same trade name on separate occasions will be uniform in composition.

This investigation showed that air-entraining admixtures could be classified into a limited number of types, each of which contained major constituents that could be fairly easily determined in the laboratory. It is concluded that the quantitative determinations of these constituents can be used as means for adequately ascertaining the uniformity of different lots of the same materials, something which cannot be accomplished by short-term physical tests as now applied.

A qualitative scheme for the classification of unknown materials as to the type or types of compounds present was developed as a part of this investigation. The principles applied in making the quantitative determinations of the various constituents are discussed and the methods that are suitable for use in each case are given.

THE CHANGES in properties caused by entrained air in portland cement concrete have been studied extensively and information is generally available to show the immediate effects to be expected for a given amount of entrained air. Since most of this work has been done with only three approved admixtures, each of which is a proprietary product, the longtime effects of these particular materials on the strength and durability of the concrete are also well established. Recently, however, the number of air-entraining admixtures on the market has increased greatly and, while their ability to entrain air in the concrete and their initial effect on its strength can be determined within a relatively short time, the demonstration of their effect on longtime strength as well as on durability requires much more extensive testing.[1]

The American Society for Testing Materials has adopted a tentative specification for air-entraining admixtures, C 260-52T, and a method of test, C 233-52T(*1*).[2] In effect, this specification and method of testing describe a lengthy program of evaluation of the admixture by comparing most of the important properties of concrete specimens made both with and without the admixture. Because of the length of time and expense involved in such a program, however, it is impracticable to apply as a control measure. Accordingly, once a product has been shown to be satisfactory, the general policy of many consumer organizations is either to approve subsequent use of the material without further tests or to require only tests to determine ability to entrain air and effect on strength.

Unfortunately, even when the latter practice is followed there is no assurance that materials supplied at different times under the same brand name will be uniform in composition, since the results of limited performance tests will not necessarily reflect changes in the composition of a product. Appreciable alteration in the composition of a product could affect its influence on the strength and durability of the concrete and thus it is extremely desirable to obtain some assurance that each lot of material will have substantially the same composition as that initially approved.

Chemical Analysis Approach

Some scheme of chemical analysis would appear to be the most direct approach to this end, but the complex chemical nature of the admixtures on the market has generally discouraged the development of laboratory methods for the analyses of most of these materials. It has been generally considered that such tests would be too involved and time consuming for practical applications. However, a study of the literature and the results of chemical analyses of a wide variety of commercial admixtures have shown that, despite the complexity of these materials, they are derived from a limited number of sources and can be chemically differentiated into a few basic types. It was also found that relatively simple tests are available to the analyst for qualitatively identifying the type and for quantitatively determining one or more of the chief constituents of each type. Thus, while chemical tests alone cannot be used as an indication of the ultimate behavior of an admixture in concrete, they can be used to give the consumer some assurance of the uniformity of different lots once a material has been proved satisfactory.

This article presents a scheme of identification and methods of analysis which are applicable to various types of admixtures. Also included is a discussion of the sources of these various types. The data presented are based on the results of chemical analyses made on 28 samples of commercially marketed materials from 19 different manufacturers. At the time they were obtained, these materials were considered to be representative of all the various types on the market. New materials which have appeared since these were first obtained may not necessarily be of the same types but it is believed that, even where new types have been introduced, somewhat the same general procedures will be applicable with slight modifications and additions of definite identification tests.

Sources of Admixtures

The primary action of any concrete admixture used chiefly for air entrainment is that of a foaming agent. Thus, they are surface-active agents and as such are closely allied to the products of the detergent and emulsifier industries. Many of the materials marketed as concrete admixtures are obtained as byproducts of various industries or have been developed from materials which were originally classified as waste. The materials tested were found to originate from five major sources: petroleum refining processes, paper pulp manufacture, wood stump processing (in the

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[1] A companion article in this issue presents an evaluation of the same air-entraining admixtures that are considered here. The samples are identically numbered in both cases. In this article the samples are grouped according to treatment for chemical analysis, but in the evaluation study they are grouped according to source.

[2] Italic numbers in parentheses refer to the list of references in appendix III, p. 277.

Table 1.—Qualitative identification of main constituents in air-entraining admixtures

Add water to original material; shake; note solubility.

If completely soluble, see part A. If emulsion forms, see part B. If an insoluble phase or deposit forms, see part C.

A. Completely soluble: Acidify water solution with hydrochloric acid. Note presence of separation.

- Fatty layer separates: Test water solution of original material for ammoniacal nitrogen (test 2) and albuminoid nitrogen (test 1). Test ash of original material for cations (sodium, potassium, and calcium) and anions (sulfate, chloride, and carbonate). Extract the fatty layer with chloroform and dry. Saponify a portion with alcoholic potassium hydroxide, evaporate, dry, and ignite. Test this ash for sulfate.
  - Sulfate present: Indicates a salt of a sulfonated hydrocarbon.
  - Sulfate absent: Indicates a soap. Make Liebermann-Storch test (test 6) on portion of chloroform extract.
    - Positive result: Indicates rosin or pine wood resin soap.
    - Negative result: Indicates naphthenic or fatty acid soap.
    - Determine solubility of the chloroform extract in ligroine (petroleum ether) or gasoline.[1]
      - Substantially soluble: Indicates rosin soap.
      - Substantially insoluble: Indicates resin soap.
      - Easily soluble: Indicates fatty acid soap.
      - Difficult to dissolve or insoluble: Indicates naphthenic acid soap.
- No definite separation: Make Biuret test (test 5) on original material.
  - Positive result: Indicates proteinaceous acids or salts. Ignite original material.
    - Ash prominent: Indicates proteinaceous salts of metals. Test for anions and cations (especially calcium).
    - Ash negligible: Indicates proteinaceous acids.
  - Negative result: Test ash of original material for sulfate.
    - Sulfate present: Indicates sulfonated lignin salts, sulfonated alkyl aryl salts, or inorganic sulfate. Test original material with permanganate (test 7).
      - Color not discharged immediately: Indicates alkyl aryl sulfonate salt and/or inorganic sulfates. Determine solubility in 95-percent alcohol.
        - Completely soluble: Indicates alkyl aryl sulfonate without builder.
        - Completely insoluble: Indicates inorganic sulfates. Test for anions and cations.
        - Partially soluble: Indicates alkyl aryl sulfonates with builder. Test insoluble for anions and cations.
      - Color discharged immediately: Indicates sulfonated lignin salts. Test aqueous solution of original material for anions and cations.
    - Sulfate absent: Such material not included in this study.

B. Emulsion forms: To the original material add sufficient alcohol and water to obtain a 50-percent alcohol-water solution. Extract with ligroine or ethyl ether.

- With the alcohol-water phase, remove the solvent by evaporation. A substantial amount of extract indicates soaps or sulfonated hydrocarbon salts. Examine by procedures for completely soluble material in part A.
- With the ligroine or ether phase, evaporate the solvent. Add an aqueous solution of sodium hydroxide; shake. (Heat to saponify if necessary.)
  - Completely miscible: Indicates presence of unsaponified organic acids in original material.
  - Partially miscible: Indicates presence of both free acid and unsaponifiable matter in original material.
  - Completely immiscible: Indicates unsaponifiable matter without free acids in original material.

C. Insoluble phase or deposit: Add water and chloroform to the original material, shake, and note solubility.

- Insoluble matter still evident: Make Biuret test (test 5) on original material.
  - Positive result: Indicates glue. Run molybdate test (test 4) or tannic acid test (test 3) on original material.
    - Positive test: Substantiates presence of glue. Determine solubility of dried original material in 95-percent alcohol (cold).
      - Partially soluble: Indicates other organic material also present. Test ash of alcohol-soluble matter for sulfate.[2]
      - Not soluble: Indicates no other type of material present.
    - Negative test: Such material not included in this study.
  - Negative result: Such material not included in this study.
- Completely soluble: Indicates presence of soap-forming acids and/or unsaponifiable matter. Saponify original material by refluxing with alcoholic potassium or sodium hydroxide. Add water and extract with ligroine or ethyl ether. Separate phases and dry.
  - Material in water-alcohol solution indicates original presence of soap-forming acids. For further identification, test according to procedure in part A.
  - Material in ligroine solution indicates unsaponifiable matter. Further tests should be quantitative.

[1] When mixtures of soaps are present, solubility characteristics of the chloroform extract in ligroine will not be conclusive. In such cases, quantitative determinations must be used to identify the various classes definitely.

[2] Some products contain alkyl aryl sulfonates in addition to glue.

naval stores industry), animal hide processing, and the processing of fats and oils. The manner in which these materials are obtained from each of these industries is discussed in appendix I (p. 274), since it is believed that a general knowledge of the derivation of these products not only will be of interest to the engineer but also will aid the chemist in evaluating the results of his tests.

Included in the five major sources mentioned are the three materials which have been used most extensively for air entrainment up to the present time—neutralized resin extracted from pine stumps, triethanolamine salt of a sulfonated hydrocarbon, and material composed essentially of calcium lignosulfonate (*2*). Other materials which have been specifically reported in the literature as having been successfully used include sulfonated oils, fatty acids and soaps, and sodium lauryl sulfonate (*3*), waste paper pulp sulfite liquors and sodium abietate (*4*), sulfated fatty alcohols, sodium alkyl aryl sulfonates with and without sodium carboxymethylcellulose and animal glue (*5*), aluminum powder (*6*), rosin and rosin soaps, vegetable and animal fat, and naphthenic acids (*7*), complex nitrosyl sulfonates (*8*), and polyethylene glycol ethers and esters (*9*). Most of these materials are products of the five major sources mentioned and, therefore, can be identified and analyzed by the methods presented.

Chemical Analysis

The procedures to be used for examining the different admixtures in the laboratory depend upon the basic chemical type of organic material present rather than the industrial source of the material. The methods of analysis are based on this consideration and the materials are grouped according to the type of constituent present. Seven types were represented by the admixtures examined: soaps, synthetic detergents, sulfonated lignins, sulfonated hydrocarbons, proteinaceous materials, emulsifiable mixtures, and soap-forming acids.

Some manufacturers supply sufficient information so that the type or types of compounds present in their product can be surmised without qualitative tests, but often a material is received in the laboratory for which such information is not known. In such cases, qualitative tests must be conducted. There are several qualitative and quantitative methods of examination that cover a wide variety of wetting, dispersing, and emulsifying agents which may be used to advantage *(10, 11, 12)*. In all cases, however, they do not adequately cover the present field of commercial admixtures.

Table 1 presents a scheme of identification which is believed to be adequate for all the types of materials examined in this investigation. This scheme was evolved from the actual results of the qualitative tests made, and is based on the consideration of the presence of only one major type of material except where combinations are normally expected. The presence of one type of material does not necessarily imply the absence of any other. Thus, for complete information where positive indications are obtained, tests under the negative indications must also be run. For mixtures of two or more types, some of the tests listed will be inconclusive. Despite these shortcomings, however, this systematic approach to the classification of unknown materials will save much time and give the required information for most of the commercial admixtures on the market at the present time.

Details of the various tests indicated by number in table 1 and also in the subsequent discussions are given in appendix II (p. 276).

Analysis of Soaps

The analytical procedures presented for the soap group are applicable to fatty, rosin, resin, and naphthenic acid soaps of alkalies, ammonia, or amines. Unsaponified matter may or may not be present. Of these materials, the rosin and fatty acid soaps are usually referred to as "true soaps," whereas the others are often designated as salts of the particular type of carboxylic acid concerned.

Eleven of the admixtures examined are in this group and the results of the tests on these materials are shown in table 2. Samples 1–6 are aqueous solutions of sodium soaps of resins extracted from wood. Sample 7 contains the same type material as samples 1–6 but was marketed as the dried product. Sample 8 is an aqueous solution of sodium soaps of fatty acids, resin (lignin) acids, and perhaps some rosin acids. Sample 9 is an aqueous solution of triethanolamine and sodium soaps of rosin, resin (lignin), and perhaps some fatty acids. Sample 10 is an aqueous solution of ammonium and sodium soaps of rosin and resin (lignin) acids containing appreciable unsaponified matter; it probably also contains some fatty acid soaps. Sample 11 is an aqueous solution of sodium soaps of carboxylic acids (most likely naphthenic). Samples 1–7 were derived from processing pine wood stumps, samples 8–10 are probably byproducts of the paper pulp industry, and sample 11 is evidently derived from petroleum refining processes.

As indicated in table 1, all of these materials and materials of the same general type are characterized by fair solubility in water and incompatibility in dilute hydrochloric acid, the organic acids separating out of solution under such conditions.

Quantitative determinations

The quantitative determinations which are considered necessary for the basic evaluation of materials in this group are those of moisture content (tests 24 or 25), ash (test 12), and total fatty matter (test 18). The determination of moisture by the oven-drying method gives a fairly reliable measure of the concentration of inorganic and organic constituents in aqueous solutions. When analyzing petroleum products such as naphthenic soaps, however, there is some danger of volatilizing some petroleum constituents. In these cases a direct water-content determination by the distillation method is recommended. This procedure involves an azeotropic distillation using some fairly low-boiling solvent such as toluene or xylene. The ash determination represents a measure of inorganic metals or metal salts present and the

Table 2.—Analyses of soap materials

<table>
<tr><th rowspan="2"></th><th colspan="11">Identification Number</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th><th>6</th><th>7</th><th>8</th><th>9</th><th>10</th><th>11</th></tr>
<tr><td colspan="12">CHARACTERISTICS</td></tr>
<tr><td>Probable source</td><td colspan="7">Pine wood stumps</td><td colspan="3">Paper pulp byproducts</td><td>Petroleum refining</td></tr>
<tr><td>Form</td><td colspan="6">Liquid</td><td>Solid</td><td colspan="4">Liquid</td></tr>
<tr><td>Color</td><td colspan="6">Very dark brown</td><td colspan="2">Brown</td><td>Dark brown</td><td>Brown</td><td>Dark brown</td></tr>
<tr><td>Odor</td><td colspan="7">Pine oil</td><td>Wood pulp</td><td>Rosin</td><td>Ammonia</td><td>Petroleum</td></tr>
<tr><td colspan="12">QUALITATIVE TESTS</td></tr>
<tr><td>Solubility:</td><td colspan="11"></td></tr>
<tr><td>In water</td><td colspan="8">Soluble</td><td colspan="2">Turbid</td><td>Soluble</td></tr>
<tr><td>In aqueous sodium hydroxide</td><td colspan="8"></td><td colspan="2">Soluble</td><td></td></tr>
<tr><td>In dilute hydrochloric acid</td><td colspan="11">Fatty layer separates</td></tr>
<tr><td>Ash:</td><td colspan="11"></td></tr>
<tr><td>Presence</td><td colspan="11">Present</td></tr>
<tr><td>Anions present</td><td colspan="11">Carbonate[1]</td></tr>
<tr><td>Cations present</td><td colspan="11">Sodium[2]</td></tr>
<tr><td>Extracted acids:</td><td colspan="11"></td></tr>
<tr><td>Liebermann-Storch test for rosin</td><td colspan="10">Positive</td><td>Negative</td></tr>
<tr><td>Solubility in ligroine</td><td colspan="7">Essentially insoluble</td><td colspan="3">Not determined</td><td>Insoluble</td></tr>
<tr><td>Ammoniacal nitrogen</td><td colspan="8">Not determined</td><td>Negative</td><td>Positive</td><td>Trace</td></tr>
<tr><td>Albuminoid nitrogen</td><td colspan="8">Not determined</td><td>Positive</td><td>Negative</td><td>Negative</td></tr>
<tr><td colspan="12">QUANTITATIVE DETERMINATIONS</td></tr>
<tr><td>Solids ..... percent</td><td>15.3</td><td>11.3</td><td>27.6</td><td>16.2</td><td>14.9</td><td>15.6</td><td>96.9</td><td>12.9</td><td>12.4</td><td>17.9</td><td>[3] 15.2</td></tr>
<tr><td>Ash ..... do</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>2.0</td><td>.8</td><td>1.1</td><td>2.2</td></tr>
<tr><td>Sodium hydroxide ..... do</td><td>1.9</td><td>1.3</td><td>3.3</td><td>2.1</td><td>2.0</td><td>[4] 1.4</td><td>8.6</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Triethanolamine ..... do</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>2.1</td><td></td><td></td></tr>
<tr><td>Total nitrogen ..... do</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>.4</td><td></td><td></td></tr>
<tr><td>Ammoniacal nitrogen ..... do</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>.8</td><td></td></tr>
<tr><td>Fatty matter:</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Total ..... do</td><td>10.1</td><td>10.0</td><td>22.6</td><td>12.5</td><td>11.5</td><td>14.2</td><td>74.9</td><td>10.9</td><td>[5] 5.3</td><td>16.8</td><td>14.1</td></tr>
<tr><td>Rosin [6] ..... do</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[7] 86.8</td><td>31.2</td><td></td></tr>
<tr><td>Methoxyl [6] ..... do</td><td>4.9</td><td>2.1</td><td>4.2</td><td>4.6</td><td>5.3</td><td>4.4</td><td>3.2</td><td>.5</td><td>4.0</td><td>.3</td><td></td></tr>
<tr><td>Unsaponified matter [6] ..... do</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>36.3</td><td></td></tr>
</table>

[1] Sample 8 also contains sulfate.

[2] Sample 6 also contains potassium.

[3] Probably low because of volatility of petroleum constituents.

[4] Sample 6 also contains potassium hydroxide.

[5] Probably low because of emulsions formed during extraction.

[6] Results expressed as percentages of total fatty matter.

[7] Determination made on original material: the calculated percentage is therefore probably high since the total fatty matter was low.

ash should be qualitatively analyzed for the alkali and alkaline earth metals—sodium, potassium, and calcium. The anionic groups—sulfate, chloride, and carbonate—should also be determined. The term total fatty matter, as applied here, represents all the organic matter extracted by chloroform from an acidified aqueous solution of the admixture. It includes all the unsaponified and unsaponifiable constituents originally present in the material as well as the organic acids resulting from the breakdown of the soaps.

Other Tests

In order to obtain further information as to the type of soap present, additional determinations must be made, the necessary tests being dictated by the nature of the soap or soaps. At times, the odor of the material may be used as a clue to the presence of certain constituents. A strong odor of petroleum indicates a naphthenic acid soap while a pulp-like smell suggests a soap derived from paper pulp wastes. Where a fair amount of rosin soap is present, the characteristic odor of rosin usually will be evident.

As shown in table 1 (part A) when rosin is indicated to be present by the Liebermann–Storch test (test 6), further confirmation is needed inasmuch as soaps of lignin resins derived from pine wood also give a positive indication of rosin because of incomplete separation during the extraction processes. In these cases the solubility of the extracted total fatty matter in ligroine (petroleum ether) should be evaluated. Where substantial solubility exists, rosin soaps in appreciable amounts are indicated and they should be determined by the standard A.S.T.M. (McNicoll) method (test 29). If the material is essentially insoluble in ligroine and a positive Liebermann–Storch test has been obtained, a lignin-derived resin soap (pine wood extracts) is indicated. The lignin or total resin, as such, is difficult or impossible to determine directly. However, lignin is characterized by certain methoxyl groupings ($-OCH_3$) in its structure and although the amount of methoxyl is dependent upon the type of wood, the source, and method of processing, it is generally considered to be nearly constant for resins from the same source and process. Thus, a determination of this constituent (test 23) is a means of comparing materials from the same source. Where the average methoxyl content of the resins is known, the approximate percentage of the latter constituent may be calculated.

When the extracted total fatty matter gives a negative test for rosin and is essentially insoluble (or very slowly soluble) in ligroine, naphthenic acids are indicated. The quantitative procedure presented for these materials is based on the formation of their copper salts and their solubility in benzene (test 26). A negative test for rosin coupled with a high degree of solubility in ligroine is indicative of fatty acids or unsaponifiable matter. If unsaponified or unsaponifiable matter is believed to be present in the original material, they should be determined by extraction from a 50-percent alcohol-water solution with ligroine or ethyl ether (test 36). This is a measure of any free organic acids and unsaponifiable oils, fats, etc., which may be present. Fatty acid soaps are indicated by difference after determining the other constituents. They may be characterized by the iodine number (test 22) of the extracted fatty matter.

Table 3.—Analyses of alkyl aryl sulfonate detergents

	Identification number					
	12	13	14	15	16	17
CHARACTERISTICS						
Probable source	← Synthesized from petroleum distillates →					
Form	← Solid →				← Liquid →	
Color	← Cream →				← Pale yellow →	
Odor	← Petroleum →					
QUALITATIVE TESTS						
Solubility in water	← Soluble →					
Aqueous solution:						
Compatibility with dilute hydrochloric acid	← Compatible →					
Permanganate test	← Does not decolorize rapidly →					
Alcohol-soluble material:						
Anions in ash	← Sulfate →					
Cations in ash	← Sodium →					
Alcohol-insoluble material:						
Presence	← Present →				Trace	Present
Anions present	← Sulfate →					Sulfate
Cations present	← Sodium →					Sodium
QUANTITATIVE DETERMINATIONS						
Water percent	2.3	3.9	2.5	6.3	63.5	63.4
Alcohol-insoluble do	53.1	10.1	59.9	13.6	.1	4.0
Fatty matter do	3.1	1.4	2.1	1.6	.6	.4
Alkyl aryl sodium sulfonate [1] do	41.5	84.6	35.5	78.5	35.8	32.2
Alkyl aryl sodium sulfonate, factor [2]	11.9	23.7	10.0	20.9	9.0	8.4

[1] By difference.

[2] Milliliters of 0.1N sodium hydroxide required to neutralize reaction products of 1 gram of original material.

For materials containing mixtures of these soaps, the qualitative differentiation based on the Liebermann–Storch test and relative solubility in ligroine will be inconclusive and all the quantitative tests must be run for complete information.

Ammonium and amine soaps are often found in commercial admixtures and, since the ash will give no indication of these materials, tests must be made on the original admixtures to determine their presence. Ammonia, both in the free state and as ammonium salts, is detected by odor or a litmus paper test (test 2) and when present should be quantitatively determined (test 11). Amine soaps are indicated by a positive test for albuminoid nitrogen (test 1). Most soaps of this type are prepared with triethanolamine and this constituent can be quantitatively determined as the alcohol-insoluble portion of the dried aqueous extract after acidification (test 34). The weighed residue should be the hydrochloride salt of triethanolamine and it can be confirmed by determining its melting point (test 8).

Synthetic Detergents

The materials included in the synthetic detergent group are mainly limited to the petroleum derived alkyl aryl sodium sulfonates where there is only one alkyl and one aryl group present. The aryl group is either a benzene or naphthalene ring and the alkyl group is usually obtained from a petroleum distillate such as kerosine. Large amounts of sodium sulfate and smaller amounts of sodium chloride and mineral oils are also usually present as a consequence of the processing techniques.

The materials of this type marketed as admixtures are similar to the household detergents used for laundering and dishwashing. They are sold both as solid products and in aqueous solutions. Six of the materials examined were of this type and the results of the tests made are given in table 3. Samples 12–15 were solids containing considerable sodium sulfate, sample 16 was an aqueous solution of an alkyl aryl sulfonate without inorganic salts, and sample 17 was an aqueous solution containing some sodium sulfate.

As indicated in table 1, these materials are characterized by solubility in both water and dilute acid solution. They usually have a characteristic petroleum odor. Organically bound sulfur and sodium in the alcohol-soluble extract (which can be detected by examining the ash of this extract) are also indications of the presence of these synthetics.

The determinations considered to be necessary for the basic evaluation of these materials include those of water content (test 24), fatty matter (test 19), inorganic chlorides and sulfates, and a determination of the alkyl aryl sodium sulfonate (tests 32 or 33). The direct determination of water content by the distillation method should always be used for these materials since the organic matter tends to be partially decomposed and volatilized at the temperature used in the oven-drying test. The fatty matter is a measure of free oils and other hydrocarbons which are present mainly as impurities resulting from the manufacturing process. When inorganic chlorides or sulfates are shown to be present by qualitative tests on aqueous solutions of the original material, they must be taken into account in calculating the alkyl aryl sulfonate by difference. The sulfate

Table 4.—Analyses of sulfonated lignin salts

	Identification number				
	18	19	20	21	22
Characteristics					
Probable source	← Paper pulp byproducts (acid process) →				
Form	← Powdered solid →				
Color	← Tan →				
Odor	← Slightly pungent →				
Qualitative Tests					
Solubility:					
In water	← Soluble →				
In alcohol, xylene, and chloroform	← Insoluble →				
Permanganate test	← Decolorizes immediately →				
Aqueous solution:					
Compatibility with hydrochloric acid	← Compatible →				
Test for calcium	← Positive (predominant) →				
Test for sodium	← Positive →				
Test for potassium	← Positive →				
Test for chloride	Trace	← Positive →			
Test for sulfate	← Negative →				
Ash:					
Presence	← Present →				
Test for sulfate	← Positive →				
Test for calcium	← Positive →				
Quantitative Determinations					
Water .... percent	14.1	21.2	19.0	23.6	22.3
Calcium chloride .... do	.5	35.5	27.5	35.0	57.3
Calcium lignosulfonate [1] .... do	85.4	43.3	53.5	41.4	20.4
Methoxyl [2] .... do	9.4	8.8	9.5	9.4	7.8

[1] By difference. [2] Based on calcium lignosulfonate content.

(usually the sodium salt) is insoluble in alcohol and thus an alcohol extraction (test 9) serves adequately to determine this constituent. However, sodium chloride is soluble in alcohol and it must be calculated by determining the chlorides present in the alcohol-soluble extract (test 17).

The accuracy of the result obtained for synthetic detergent computed by difference (test 32) suffers from all the deficiencies inherent in any such calculation. An alternate and more direct measure of the synthetic detergent is obtained by a fairly rapid volumetric procedure (test 33). In this method, the alkyl aryl sulfonate is made to react with paratoluidine hydrochloride, the product extracted with carbon tetrachloride and subsequently titrated with standard sodium hydroxide. The results are usually expressed as a factor, i.e., the milliliters of 0.1-normal sodium hydroxide required to neutralize the reaction product from 1 gram of sample. Expressed in this form, the results offer a reliable and precise means for determining the uniformity of different lots of the same material, but no comparison between different materials can be made and the exact percentage of the constituent cannot be calculated. However, if a standard sample of the material being examined is available, the solution can be standardized in terms of the standard and the results expressed as percentages.

Sulfonated Lignin Materials

The materials in the sulfonated lignin group are mainly the calcium salts of sulfonated lignin derived from the acid (sulfite) process of paper pulp manufacture. These materials are commonly referred to as calcium lignosulfonates. Some commercially marketed materials also contain calcium chloride and small amounts of alkyl aryl sulfonates.

These materials are usually available as tan powdered solids and have a pungent odor reminiscent of wood pulp. Five of the admixtures examined were of this type and the results of the tests made on these materials are shown in table 4. Sample 18 was chiefly calcium lignosulfonate and water, while samples 19–22 contained considerable calcium chloride in addition to the lignosulfonate.

As indicated in table 1, these admixtures are characterized by a high solubility in water, and compatibility with dilute mineral acids. They are mostly insoluble in organic solvents. Since the sulfur is organically bound, the material gives a negative test for sulfate in aqueous solution, and a positive test in the ash after igniting at 600° C. This type of lignin-derived material is easily oxidized and, therefore, immediately decolorizes an acidified permanganate solution (test 7). As shown in table 1, this test is a basis for distinguishing between these materials and the alkyl aryl sulfonates which do not rapidly decolorize the permanganate solution.

The determinations considered necessary for the basic evaluation of these materials are those of moisture content (test 24), calcium chloride if present (test 14), and calcium lignosulfonate by difference (test 16). Even though the materials are usually solid powders, the water content is normally appreciable. Oven drying is not advisable because of the possible decomposition on heating, and therefore the distillation method is recommended. In this connection, triethylbenzene has been suggested by some as the solvent to be used for these materials. However, this solvent was found to be undesirable in that its higher boiling temperature caused some decomposition resulting in much "frying" and "bumping" during the distillation. Toluene or xylene were found to be much more satisfactory. The water-soluble chloride ion is determined by titration with standard silver nitrate solution, and calculated as calcium chloride. Calcium lignosulfonate is then generally considered to be the difference.

This procedure, while adequate for materials from known sources, does not give complete information on new or unknown materials since inert ingredients, if present, would be included as calcium lignosulfonate. Thus, it is suggested that the methoxyl determination (test 23) could be applied to these materials in much the same manner as for the lignin-derived resins. While extensive data are not available, all the materials of this type examined had approximately the same methoxyl contents, based on the calcium lignosulfonates. When available, a spectrophotometer can be used to determine lignosulfonate (test 15). This procedure involves the use of nitric acid to oxidize the lignin-derived material, and the developed color is measured photometrically. However, this test was not made in the Bureau laboratory.

A number of the commercial admixtures of this type contain small amounts of alkyl aryl sulfonates. These small amounts are not detectable by the usual qualitative tests and the volumetric determination described for the synthetic detergents is not suitable. In cases where they are known or believed to be present, they can be determined spectrophotometrically by the methylene-blue method (test 31).

Salts of Sulfonated Hydrocarbons

The tests for salts of sulfonated hydrocarbons are applicable to alkali, ammonium, or amine salts of sulfonated hydrocarbons derived from petroleum. The materials considered here are those obtained from the so-called "petroleum sulfonates" which result from acid treatments during the petroleum refining processes. They have several adjacent benzene nuclei rather than a single benzene or naphthalene ring common to the synthetic detergents which have been considered as a separate group.

The only admixture examined which was classified as being of this type was sample 23, the results being shown in table 5. This was an aqueous solution of a proprietary material commonly referred to as a triethanolamine salt of sulfonated hydrocarbons. This type of material has been extensively used both as a concrete admixture and a cement additive.

These materials are somewhat similar to the soaps in their qualitative characteristics and therefore require somewhat the same determinations. They are soluble in water and show some incompatibility in acidified aqueous solutions. Usually, more highly acidified solutions are required to detect incompatibility than is required in the case of the soaps. As indicated in table 1, the presence of the organically bound sulfur in the hydrocarbon portion of the molecule is the best means of differentiation from the soaps.

Table 5.—Analysis of a sulfonated hydrocarbon salt

	Sample 23
CHARACTERISTICS	
Probable source	Petroleum refining
Form	Liquid
Color	Dark brown
Odor	Petroleum
QUALITATIVE TESTS	
Solubility in water	Soluble
Compatibility of aqueous solution with hydrochloric acid	Incompatible
Ash:	
Presence	Present
Test for sulfate	Positive (predominant)
Test for sodium	Positive
Test for potassium	Positive
Ammoniacal nitrogen	Negative
Albuminoid nitrogen	Positive
QUANTITATIVE DETERMINATIONS	
Solids percent	12.5
Ash do	2.4
Total fatty matter do	6.8
Triethanolamine do	4.2
Total nitrogen do	.5

The determinations considered necessary for the basic evaluation of these materials are essentially the same as for the soaps. These are a determination of moisture content (tests 24 or 25), an ash determination (test 12), and a determination of the total fatty matter (test 18). As is true for all the petroleum-derived materials, the distillation method for water content is generally preferred because of the volatility of some of the constituents, but for fairly dilute solutions, such as sample 23, where unusually small amounts of material must be used in the distillation method, this latter method is at a disadvantage. The ash is important in determining the amounts and types of metallic cations present. Since the triethanolamine salts are very frequently used, the test for albuminoid nitrogen (test 1) is especially important for these materials and should always be conducted even when a fair amount of ash is present. Where a quantitative determination of triethanolamine is indicated to be necessary, it is conducted in the same manner as for soap materials.

In the determination of the total fatty matter the procedure varies slightly from that used for the true soaps in that a high concentration of mineral acid must be maintained not only in the initial solution but also in the subsequent wash water. This is necessary because the organic sulfonic acids are appreciably soluble in pure water but are "salted out" from the acid solution. For sulfonated hydrocarbons from the same source, the amount of organically bound sulfur should maintain a substantially constant relationship with the total fatty matter, and thus the quantitative determination of these sulfonates, as measured by the sulfate in the ash (test 30), may be used as a supplementary test for gauging uniformity of supply.

Proteinaceous Materials

Admixtures considered in the proteinaceous materials group include salts of proteinaceous acids, and glue or other proteins, any of which may be admixed with non-proteinaceous substances.

Three of the admixtures examined were classified as being of this type. The results of these analyses are given in table 6. Samples 24 and 25 were essentially aqueous solutions of the calcium salts of complex proteinaceous acids obtained from protein hydrolysis. Sample 26, while classified as a proteinaceous material because of its glue content, differed considerably from the other two. This material was found to be an aqueous solution of a glue-like proteinaceous material containing a large amount of an alkyl aryl sodium sulfonate along with small amounts of sodium sulfate and petroleum oil. All of these materials give a positive Biuret test (test 5), which is indicative of proteinaceous material.

The determinations considered necessary for the basic evaluation of these materials are those of moisture content (tests 24 or 25), ash (test 12), and total nitrogen (test 27). The nitrogen is conveniently determined by a modified Kjeldahl procedure and its presence in substantial amounts is characteristic of proteinaceous materials since proteins contain from 15 to 17.5 percent nitrogen. Any additional tests to be made will depend upon the particular nature of the material. A putrefactive odor, solubility in water, and the predominance of calcium in the ash is indicative of calcium salts of complex proteinaceous acids. If this is the case, the quantitative determination of calcium in the ash is of value in determining the uniformity of different lots from the same source (test 13). The proteinaceous acids cannot be determined as such because of their complexity and, therefore, the total nitrogen content must suffice as an approximate measure of proteinaceous materials. If desired, the value for total nitrogen may be used to calculate the approximate protein content of the sample by the use of an empirical factor (test 28).

Those admixtures containing glue along with other materials (as exemplified by sample 26) must be analyzed in a much different manner than the former proteinaceous materials. The glue portions of these materials are relatively insoluble in cold water, alcohol, and chloroform. Their solubility increases somewhat in hot water or hot alcohol. There are two tests by which glue can be qualitatively indicated (tests 3 or 4). These tests are based on the precipitation of glue with tannic acid or molybdate ion.

A quantitative estimation of the glue can be made by an empirical method (test 21). This procedure assumes an average nitrogen content for all animal glue. Therefore, the total nitrogen multiplied by the average factor gives the approximate amount of glue. The loss in weight obtained by igniting the dried alcohol-insoluble material can also be considered glue when it is known that no other organic alcohol-insoluble material

Table 6.—Analyses of proteinaceous materials

	Identification number		
	24	25	26
CHARACTERISTICS			
Probable source	← Hides →		Hides and petroleum
Form	← Liquid →		
Color	← Brown →		
Odor	← Putrefactive →		Petroleum
QUALITATIVE TESTS			
Solubility:			
In water	← Soluble →		Partially insoluble
In dilute hydrochloric acid	← Soluble →		Not determined
In chloroform	← Not determined →		Insoluble
Biuret test for proteinaceous materials	← Positive →		
Tests for glue	← Not determined →		Positive
Ash:			
Presence	← Present →		
Anions present	← Carbonate →		Sulfate
Cations present	◀ Calcium and sodium ▶		Sodium
QUANTITATIVE DETERMINATIONS			
Solids percent	52.6	46.6	41.6
Ash do	7.4	9.6	----
Total nitrogen do	6.8	5.4	2.3
Proteins [1] do	42.8	34.0	----
Alcohol-insoluble material do	----	----	13.3
Ash from alcohol-insoluble material do	----	----	1.5
Fatty matter do	----	----	2.3
Glue-like material:			
Empirical [2] do	----	----	13.0
By difference do	----	----	12.0
Synthetic detergent:			
By difference do	----	----	26.3
Factor [3]	----	----	6.0

[1] Total nitrogen × 6.3. [2] Total nitrogen × 5.6.
[3] Milliliters of 0.1N sodium hydroxide required to neutralize the reaction products of 1 gram of original material.

Table 7.—Analysis of an emulsifiable mixture

	Sample 27
CHARACTERISTICS	
Probable source	Petroleum
Form	Liquid
Color	Reddish brown
Odor	Petroleum
QUALITATIVE TESTS	
Compatibility:	
With water	Emulsifies
With chloroform, xylene	Soluble
With dilute hydrochloric acid	Incompatib'e
Ash:	
Presence	Present
Anions present	Carbonate & sulfate
Cations present	Sodium
QUANTITATIVE DETERMINATIONS	
Water percent	0.5
Ash do	2.1
Total fatty matter do	92.9
Unsaponified matter do	85.5
Soaps, by difference[1] do	7.4

[1] Total fatty matter less unsaponified matter.

is present and when the inorganic salts are not volatilized during the ignition (test 20).

When alkyl aryl sulfonates are present along with glue, as was the case for sample 25, the examinations for this material can be conducted much the same as if alkyl aryl sulfonates alone were present. The alcohol-soluble portion will contain the alkyl aryl sulfonate plus any sodium chloride present, while the insoluble portion will contain the glue and the sodium sulfate. The volumetric method (test 33) for determining alkyl aryl sulfonates may be used for mixtures of this type.

Emulsifiable Mixtures

The materials included in the emulsifiable mixture group are those which contain chiefly unsaponifiable matter combined with sufficient soaps to form an emulsion when shaken with water. The soaps are usually sodium soaps of naphthenic acids but other types are possible.

Sample 27 was the only material of this type examined. The results of the tests made are given in table 7. This material was a mixture of a petroleum distillate in the lubricating oil range with sodium soaps of carboxylic acids (probably naphthenic). This material and materials of the same type are characterized by their ability to form a substantially stable emulsion when shaken with water. The addition of hydrochloric acid will usually break the emulsion and produce a separate fatty layer. The original material is completely soluble in chloroform.

The determinations considered necessary for the basic evaluation of these materials are those of moisture content by the distillation method (test 24), ash (test 12), total fatty matter (test 18), unsaponified plus unsaponifiable matter (test 36), and soaps by difference. The amount of moisture in these materials is usually very low and would be present only as an impurity. The ash is useful in determining the extent and type of inorganic materials present. Total fatty matter extracted with chloroform from an acidified solution includes the soap acids and the unsaponified and unsaponifiable matter. A determination of unsaponified and unsaponifiable matter will then enable the calculation of the soap acids by difference. Generally the former material will be the unsaponifiable petroleum distillate, which is identifiable by its odor and appearance.

Usually the estimation of soaps (strictly speaking, soap acids) by difference is sufficient for this type of material but, if it is considered necessary, the acids from the soaps can be recovered by acidifying the alcohol-water phase after extracting the unsaponified and unsaponifiable matter, extracting with ligroine or ether (or chloroform where naphthenics or resins are suspected) and evaporating the solvent. The recovered material can then be examined qualitatively and quantitatively in the manner described for the soaps. For complete information, tests for albuminoid and ammoniacal nitrogen (tests 1 and 2) should be conducted on the original material, since nonmetallic soaps of this type may be present. When positive indications are obtained for either of these constituents, the quantitative determinations can be made in the same manner as described for the soaps (tests 11 or 27).

Soap-Forming Acids

The soap-forming acid group includes the various organic acids capable of being saponified to form soaps. The marketed material also usually contains some unsaponifiable matter.

Sample 28 was the only material of this type examined, the results of the tests made being given in table 8. It was derived from the processing of vegetable oils and fats and contained a mixture of fatty acids and unsaponifiable fatty constituents. This material and others of the same type are characterized by a negligible amount of ash, along with negative tests for ammoniacal and albuminoid nitrogen.

Table 8.—Analysis of a soap-forming acid

	Sample 28
CHARACTERISTICS	
Probable source	Vegetable oils
Form	Paste
Color	Brown
Odor	Pungent
QUALITATIVE TESTS	
Solubility in water	Insoluble
Solubility in chloroform, ether, ligroine	Soluble
Ash	Trace
QUANTITATIVE DETERMINATIONS	
Water percent	0.4
Ash do	Trace
Unsaponifiable matter do	45.4
Total fatty matter do	99.6
Iodine No. (Wijs):	
On extracted fatty acids	30
On unsaponifiable matter	201

The determinations considered necessary for the basic evaluation of the materials are those of moisture content (tests 24 or 25), ash (test 12), and unsaponifiable matter (test 35). The water and ash determinations are considered necessary to show any large amount of impurities which may be present. When these determinations are made, it is usually considered unnecessary to evaluate the total fatty matter as such, since nominally the material is all fatty matter. A determination of unsaponifiable matter is important to show the extent to which non-soap-forming constituents are present. If the nature of the soap-forming acids is not known, they can be recovered from the alcohol-water phase after the extraction of the unsaponifiable matter. This recovered material can then be examined in the manner described for soaps. When the material is chiefly fatty acids, as in the case of sample 28, the iodine number of the fatty acids (test 22) can be used as a measure of the uniformity of different lots of the same material.

Appendix I.—Derivation of Admixtures from Industrial Processes

Petroleum Industry Products

The air-entraining admixture materials which were determined to be wholly or in part derived from the petroleum industry fall into three general categories: the naphthenates, the salts of sulfonated hydrocarbons, and the alkyl aryl sulfonates.

Naphthenic acids

Naphthenic acids are present in petroleum in small amounts, ranging from 0.1 to 3 percent. They are separated during the refining of various petroleum fractions such as ligroine (benzine), kerosine, and lubricating oil. These petroleum fractions are treated with strong sulfuric acid to remove aromatic and unsaturated compounds. After separating the bulk of the acid sludge, the residual oily layers are subjected to a dilute caustic soda wash which results in the formation and solution of sodium naphthenates (soaps of naphthenic acid). The aqueous layer containing these soaps is separated and used as wetting or emulsifying agents alone or incorporated with various lubricants to form special cutting or emulsifiable oils. Commercial products representing both types of such materials have been marketed as air-entraining admixtures. Chemically, the naphthenic acids are classified as cyclo-paraffins, characterized by a saturated five- or six-membered carbon ring associated with a carboxylic acid group (–COOH). The alkali soap of this acid has an alkali metal (usually sodium or potassium) substituted for hydrogen in the group (–COONa).

Sulfonated aromatic hydrocarbons

The second category of materials derived from the petroleum industry is the sulfonated aromatic hydrocarbons (or petroleum sulfonates). These materials are obtained during the refining of lubricating and white oil stocks with highly concentrated sulfuric acid. For example, in the preparation of white mineral oil for medicinal purposes, the stock oil is repeatedly treated with fuming sulfuric acid to remove substantially all the aromatic constituents in order to obtain the required colorless white oil. As a result of the high degree of sulfonation obtained in the acid treatment, considerable amounts of aromatic sulfonic acids are produced (*13*). These acids are principally of two types—the so-called water-soluble type predominantly contained in the lower acid sludge, and the so-called oil-soluble type predominantly found in the upper oil layer of the reaction mixture. The water-soluble acids found in the sludge impart a dark green color to their aqueous solution and are consequently referred to as "green" petroleum sulfonic acids. The acids left in solution in the oil impart a reddish color to the oil and for this reason are generally referred to as "mahogany" petroleum sulfonic acids.

After the separation of the oil layer from the acid sludge, the oil layer is washed with aqueous alcohol solutions to extract the free oil-soluble sulfonic acids, or neutralized with alcoholic alkali and the sulfonates extracted as their alkali salts. These materials are used primarily as emulsifying agents and have been proposed for use as anti-stripping agents in bituminous mixtures (*13*).

The water-soluble or "green" petroleum sulfonic acids are recovered from the acid sludge as their salts by a process of water-washing, neutralization, and alcoholic extraction. They are used primarily as inexpensive detergents and wetting agents and particularly as a base for air-entraining admixtures for concrete and asphalt emulsion stabilizers (*13*). Chemically, the water-soluble sulfonic acids are characterized by having a complex ring structure consisting of several aromatic and one saturated ring nuclei associated with one or two sulfonic acid groups ($-SO_3H$). In one patent, a condensed petroleum sulfonic acid is combined with a hydroxyl-alkyl-amine (e.g., triethanolamine) which replaces one half of all the available hydrogen in the sulfonic acid groups to form a salt useful as a cement additive or concrete admixture (*14*).

Alkyl aryl sulfonates

The third type of admixture derived from the petroleum industry is the alkyl aryl sulfonates. While in the broadest sense these materials are also sulfonated hydrocarbons, their manner of derivation and their characteristics differ widely from the group previously discussed. These materials cannot be strictly classified as byproducts since they are manufactured by a direct synthesis. They are commonly referred to as synthetic detergents and have found a wide acceptance under various trade names as household and industrial detergents. They are usually prepared by chlorinating the kerosine fraction of petroleum distillates, condensing this product with an aromatic hydrocarbon (e.g., benzene or naphthalene), and sulfonating the aromatic portion of the complex with fuming sulfuric acid. This procedure produces a complex material consisting of a central aryl (aromatic) nucleus associated with two separate side groupings. One grouping is the alkyl side chain (hydrocarbons from the kerosine fraction) and the other is the sulfonic acid group ($-SO_3H$). The product is then neutralized with aqueous sodium hydroxide to form the alkyl aryl sodium sulfonate where the sodium ion replaces hydrogen in the sulfonic acid group to form the salt ($-SO_3Na$).

It is this complex organic salt which comprises the active ingredient of this type of synthetic detergent. As a result of the presence of sulfuric acid at the time of the alkali neutralization, a substantial amount of sodium sulfate is also formed. In many applications this salt is not removed since it increases the detergency of the product. Various dry formulations vary from 85 percent alkyl aryl sodium sulfonate and 15 percent sodium sulfate to 40 percent alkyl aryl sodium sulfonate and 60 percent sodium sulfate (*15*). The material is also marketed in aqueous solutions for air-entrainment purposes. The alkyl aryl sulfonates are sometimes added in small amounts to other types of concrete admixtures to enhance their ability to entrain air.

Paper Pulp Manufacture Products

There are two general types of processes used in the manufacture of paper pulp, both of which yield byproducts suitable as such or as a base for concrete admixtures—the acid process (sulfite) and the alkaline process (sulfate and soda).

Acid process

In the acid process the wood chips are digested under pressure in a solution of sulfurous acid and calcium acid sulfite. This digestion leaves the cellulose of the wood unaffected but decomposes the lignin and complex carbohydrates originally present. After a suitable digestion period the sulfite liquor is drawn off, leaving the cellulose pulp. The sulfite liquor contains about 8 to 14 percent solids which consist of approximately 60 percent calcium lignosulfonate, 20 percent reducing sugars, and 10 percent carbohydrates, the remainder being volatile acids, methyl alcohol, acetone, and inorganic impurities (*16*). The sulfite liquors have been used directly as admixtures for concrete. The liquor is also utilized for the production of alcohol by fermenting the sugars present. Following this fermentation, or as a separate process, the liquor may be treated with excess lime in order to obtain calcium sulfite which is returned to the digestion process. The basic calcium lignosulfonate resulting from the latter operation is removed as a filter cake and pressed to reduce its moisture content to about 50 percent.

The calcium lignosulfonate thus obtained and further dried exhibits excellent dispersing properties and is used to some extent as an admixture for concrete. In one patent an admixture is described which consists mainly of the solid residue of waste sulfite liquors after they have been fermented and the alcohol removed (*17*). Undoubtedly, the major portion of these solids is calcium lignosulfonates with most or all of the sugars removed. Chemically the lignosulfonate is believed to be a polymerized material made up of numerous benzene rings with hydrocarbon side chains, methoxyl ($-OCH_3$) groups, hydroxyl ($-OH$) groups, and calcium sulfonate groups.

Alkaline process

There are two major methods of alkaline digestion of wood for pulp manufacture—the soda and sulfate processes. In the former, the wood chips are digested with sodium hydroxide, while the latter process makes use of sodium sulfate in combination with sodium sulfide. The "black liquors" obtained in both cases are somewhat similar and have been used as such or with some refinement and modification as air-entraining admixtures for concrete.

The black liquors consist of a mixture of sodium salts of straight chain carboxylic acids, resin acids (predominately abietic), sterols, alcohols, unsaponifiable matter, and inorganic salts. The crude soap skimmings from the raw liquor may be used as an emulsifier (*18*). The active materials serving this purpose are primarily the sodium soaps of the fatty and resin acids. The black liquors are also refined to yield such products as tall oil, rosin and rosin soaps, turpentine, wood alcohol, and lignin. Tall oil and rosin are used as additional sources of air-entraining admixtures. Crude tall oil is the residue obtained by evaporating the sulfate liquor and acidifying the thick curds. It is an oily resinous mixture containing 30 to 59 percent fatty acids, 34 to 64 percent resin acids, and 5 to 10 percent unsaponifiable matter (*19*). Soaps usable as admixtures can be formed by reaction of the original tall oil with an alkaline substance (e.g., caustic soda, ammonia, etc.). Rosin and fatty acids may be separated from the tall oil and these can be saponified to produce a wetting agent (*20*). Lignin isolated from these liquors also has been recently contemplated for use as a dispersing agent.

Wood Stump Processing Products

In connection with the manufacture of naval stores, pine tree stumps which have been aged in the ground for about 10 to 15 years have been found to be an economical source of turpentine, pine oil, rosin, and other special resins. These latter resins are of a lignin nature, and when neutralized have been widely used for air-entrainment purposes.

The methods used for processing wood stumps vary considerably. One patented procedure (*21*) makes use of a coal tar hydrocarbon (such as benzene, toluene, etc.) to extract the wood chips. If the chips have not been previously steamed, the extract will contain turpentine, pine oil, FF-grade rosin and other resins. The extract is drawn off and the solvent recovered by distillation. The distillation is continued to remove turpentine and pine oil. The residue, a solid containing rosin and other resins, is then dissolved in a mixture of a light petroleum hydrocarbon such as gasoline and some other suitable solvent such as furfural. The two solvent layers separate upon cooling. Rosin is obtained from the petroleum solvent and the resins are obtained from the furfural solvent. After removal of the solvent, the dried resin may be neutralized with sodium hydroxide to produce a sodium resinate which is used extensively as an air-entraining admixture for concrete. The structural chemistry of the original resin is not well developed. It is believed to be mainly of a lignin nature containing methoxyl ($-OCH_3$) and hydroxyl ($-OH$) groups as well as oxidized resin acids, oxidized terpenes, polyphenols, and polymerized terpenes (*22*). The empirical formula of the main constituent in the resin has been reported to be approximately $C_{27}H_{30}O_5$ and to have an average molecular weight of about 450 (*23*). The unneutralized resin is soluble in alcohol and coal tar solvents, but substantially insoluble in petroleum solvents and water.

Animal Hide Industry Products

Animal skins are mainly composed of proteins which nature utilizes as emulsifying agents. Thus, many byproducts incidental to the manufacture of leather also have good emulsifying properties and have been used as air-entraining admixtures.

Prior to tanning, the hides are dried, treated with a preservative (usually sodium chloride), and shipped to the tannery. The skins are then soaked in water to remove the preservative and soften the hide. This treatment also results in the solution of certain proteins and their degradation products (such as albumen). Degraded albumen contains lysalbinic and protalbinic acids which, when neutralized with some alkaline material, become useful as dispersing agents (*24*). After removing the areolar tissue, the skin is subjected to a lime-water treatment in order to hydrolyze and remove the keratinous proteins (the hair, nails, and horns of the epidermis). This results in the solution of calcium salts of complex amino acids which are also useful as dispersing agents. In addition, some of the albumen that was unaffected by the earlier water treatment is hydrolyzed and dissolved to yield more calcium salts of lysalbinic and protalbinic acids.

Gelatin, or glue, derived directly from the alkaline treatment of animal hides, is an excellent emulsifying and dispersing agent and has been used as a major ingredient in preparing commercial admixtures for concrete. One patent describes a mixture of animal hide glue and alkyl aryl sulfonate salt as being particularly suitable for producing cellular concrete (*25*).

Proteins are complex organic materials composed of units of amino acids having the general formula $NH_2-R-COOH$, where R stands for a complex carbon chain, and NH_2 and COOH the amino and carboxylic acid groups, respectively. Animal glues are merely colloidol suspensions of protein molecules in water.

Fat and Vegetable Oil Processing Products

Animal and vegetable fats and oils are hydrolyzed by various methods to produce fatty acids. The raw fats and oils derived from animal and vegetable matter may be hydrolyzed at high pressure with lime or magnesia, or they may be processed by the use of sulfuric acid and an aromatic hydrocarbon such as naphthalene. The crude fatty acids so produced may then be distilled under vacuum to separate the individual acids.

Chemically, the fatty acids are characterized by a long hydrocarbon chain which is combined with a carboxylic acid group ($-COOH$). When saponified with an alkali, the fatty acids are converted to conventional soaps. These soaps are, of course, the most widely used of all wetting agents and detergents, and are sometimes used as admixtures for entraining air in concrete. Fatty acid soaps are also sometimes found combined with other type materials in commercial admixtures either purposefully or as a natural result of processing.

Appendix II.—Test Procedures

Many of the tests referred to in the preceding text are standard procedures and are given in the American Society for Testing Materials (A.S.T.M.) Book of Standards and other publications. Others are modifications of standard procedures or tests developed during the course of this work, while still others are non-standard tests suggested by the literature. In this appendix, all of these tests are numbered and are listed alphabetically according to the constituent being determined or according to the usual designation for the test. Qualitative and quantitative tests are given under separate headings.

For the standard tests and such tests where only minor modifications were made, the A.S.T.M. Book of Standards *(26)* designation is given along with a statement of the modification if any. For all tests not included in the A.S.T.M. Book of Standards, the basic procedures, reagents, and equipment are given, with references to the orginal sources when available.

Qualitative Tests

(1) *Albuminoid nitrogen (amines, etc.)*.—After boiling the material for several minutes as shown in the test for ammonia (test 2), cool the flask and its contents to room temperature, add an alkaline solution of potassium permanganate, boil, and test the vapors with moistened red litmus paper. A blue coloration is indicative of the original presence of albuminoid nitrogen.

(2) *Ammoniacal nitrogen (ammonia, etc.)*.—Place a small amount of the sample in a small narrow-mouthed flask. Add distilled water if necessary. If the sample is acid to methyl orange, add sufficient magnesium oxide to make alkaline. Boil and test the vapors with moistened red litmus paper. A blue coloration indicates the presence of free ammonia, ammonium soaps or salts.

(3) *Glue (tannic acid test)*.—Boil several grams of tbe material with 10–15 ml. of water for a few minutes. Filter while hot and cool to room temperature. To a portion of the filtrate add an equal volume of a cold 10-percent solution of sodium chloride nearly saturated with tannic acid and freshly filtered. A pale grayish-yellow flocculent precipitate indicates the presence of glue.

(4) *Glue (molybdate test)*.—To a portion of the cool water extract obtained in test 3, add half its volume of neutral ammonium molybdate solution followed by a few drops of nitric acid. The formation of a white amorphous precipitate indicates the presence of glue.

(5) *Proteinaceous matter (Biuret test)*.—Mix about five drops of the original sample in solution with approximately 3 ml. of reagent consisting of one part of 3-percent cupric sulfate solution and forty parts of 10-percent sodium hydroxide solution. A violet to purple coloration is indicative of a proteinaceous material.

(6) *Rosin (Liebermann-Storch test)*.—Use A.S.T.M. Method D 803-51, sections 57 and 58 (1952, Part 4, p. 672). This test should be run on the extracted and dried organic acids or fatty matter. A fugitive violet coloration is indicative of the presence of rosin or rosin acids.

(7) *Sulfonated lignin (permanganate test)*.—Dissolve a small amount of the material in 10 ml. of water, add 5 ml. of dilute sulfuric acid, and then a few drops of 0.5-percent potassium permanganate solution and shake. Where the color is discharged immediately, lignin is indicated.

(8) *Triethanolamine, melting point of the hydrochloride salt*.—The melting point of pure triethanolamine hydrochloride is 177°C. A melting point within the range of 173 to 181°C. is sufficient evidence of the presence of this compound.

Quantitative Tests

(9) *Alcohol-insoluble matter*.—Follow the procedure given by A.S.T.M. Method D 820-46, sections 10 and 11 (1952, Part 7, p. 607) except that the crucible and contents should be dried at 105°C. for several hours and weighed as alcohol-insoluble matter.

Where glue is known to be present, make the extraction at room temperature in a platinum or porcelain dish. After thorough stirring with each portion of alcohol, allow the insoluble matter to settle and filter the supernatant liquid through a fritted glass crucible. When the extraction is complete, return the contents of the crucible to the evaporating dish with a stream of 95-percent alcohol. Evaporate the alcohol on a steam bath and dry the contents of the dish to constant weight at 105°C. Weigh and calculate as the percentage of alcohol-insoluble matter in the sample.

(10) *Alcohol-soluble matter*.—Use A.S.T.M. Method D 820-46, sections 10 and 11 (1952, Part 7, p. 607).

(11) *Ammoniacal nitrogen*.—Weigh accurately 1 to 3 g. of sample into a 500-ml. distilling flask. Add 200 ml. of water and 2 g. or more of magnesium oxide free from carbonates. Connect the flask to the condenser with a Kjeldahl connecting bulb, distill 100 ml. of liquid into a measured quantity of standard acid (0.1N sulfuric acid recommended) and titrate with standard alkali solution (0.1N sodium hydroxide recommended) using methyl red as an indicator. The amount of standard acid to be added depends on the expected ammonia content of the sample. Calculate the result as the percentage of ammoniacal nitrogen present in the sample (ref. *27*, method 2.27, p. 14).

(12) *Ash*.—Weigh sufficient sample into a small porcelain evaporating dish, dry over a steam bath, char and ignite to constant weight at 600°C. Calculate the results as the percentage of ash in the sample. If desired, the sample from the moisture determination (oven method) may be used for this test.

(13) *Calcium (in ash)*.—Analyze the material from the ash determination (test 12) for calcium by any suitable procedure (e.g., precipitation as oxalate, ignited and weighed as calcium carbonate or calcium oxide) after preliminary removal of silica and the ammonium hydroxide group. Express the result in terms of percentage of calcium in the ash.

(14) *Calcium chloride (Mohr's titration for chloride)*.—Weigh 1 g. of sample into a 300-ml. porcelain casserole, dissolve in 150–200 ml. of distilled water. Add 3 ml. of 5-percent potassium chromate solution and titrate with 0.1N silver nitrate solution. The end point is marked by the transition in color from a lemon yellow to a faint orange tinge which does not disappear on stirring. Standardize the silver nitrate solution with 0.1 to 0.2 grams of fused sodium chloride, titrated in the same manner as the sample. Express the result as the percentage of calcium chloride in the original sample.

(15) *Calcium lignosulfonate (colorimetric method)*. —Dissolve exactly 3.0 g. of sample in exactly 1 liter of water. Treat a 25-ml. aliquot of this solution with 5.0 ml. of concentrated nitric acid and dilute to exactly 50 ml. Measure the color developed by oxidation with nitric acid with a spectrophotometer at a wavelength of from 485 to 550 millimicrons and determine the concentration by use of a calibration curve prepared with a reference sample.

(16) *Calcium lignosulfonate (difference method)*.— Calculate the percentage of calcium lignosulfonate in the sample by subtracting the total of the percentages of calcium chloride, moisture, and synthetic detergent from 100.

(17) *Chlorides in alcohol-soluble matter*.—After the determination of fatty matter in synthetic detergent (test 19), use A.S.T.M. Method D 820-46, sections 19–21 (1952, Part 7, p. 610). Express the result as the percentage of sodium chloride in the sample.

(18) *Fatty matter (total)*.—Weigh accurately sufficient sample to yield about 1 g. of total fatty matter. Quantitatively transfer to a separatory funnel and dilute to 100 ml. with water. Acidify the material with sufficient hydrochloric acid to free all the fatty acids and extract with 75 ml. of chloroform. Follow with two more sucb extractions of 25 ml. each. Wash the collected chloroform extracts twice with slightly acidified (HCl) water, distill or evaporate to a small volume, and then transfer to a small tared dish. Evaporate the residue to dryness and when free of chloroform, dry at 105°C. for 15-minute periods to constant weight. Calculate the result as the percentage of total fatty matter in the sample.

Note: When dealing with salts of petroleum sulfonates (sulfonated hydrocarbons), it will be necessary to use more strongly acidified solutions in order to reduce the solubility of the organic material in the aqueous phase. If desired, substantial amounts of sodium chloride in the aqueous phase will serve the same purpose.

(19) *Fatty matter in synthetic detergents*.—After the separation of the alcohol-soluble matter (test 10), determine as directed in A.S.T.M. Method D 820-46, sections 16–18 (1952, Part 7, p. 609).

(20) *Glue (from alcohol-insoluble matter)*.—After the determination of alcohol-insoluble matter (test 9), ignite the material at 600°C. and weigh. The loss in weight represents an approximation of the glue content, which is expressed as a percentage of the original material.

(21) *Glue (from total nitrogen)*.—Multiply the percentage of total nitrogen in the sample (test 27) by the factor 5.6 to obtain the percentage of glue present. See A.S.T.M. Method D 982–52, section 7c (1952, Part 7, p. 854). The factor 5.6 is an empirical relation based on an average nitrogen content for animal glues.

(22) *Iodine number (Wijs method)*.—Use the procedure given in A.S.T.M. Method D 460-46, section 38 (1952, Part 7, p. 592).

(23) *Methoxyl content*.—Use the A.S.T.M. Method C 114-51T, sections 2–4 (1952, Part 3, p. 102), except that the result shall be expressed as the percentage of methoxyl (OCH_3) present in the total fatty matter and that the determination shall be made on the material obtained as total fatty matter (test 18). In the case of solid samples, no previous extraction of total fatty matter is necessary, and is actually undesirable in the case of lignosulfonates. For the latter materials, the results should be expressed on the basis of the calcium lignosulfonate content. 1 ml. of 0.1N sodium thiosulfate solution is equivalent to 0.000517 g. of methoxyl (OCH_3).

(24) *Moisture (distillation method)*.—Use A.S.T.M. Method D 460-46, sections 11–13 (1952, Part 7, p. 584). Calculate the result as the percentage of moisture or water in the sample.

(25) *Moisture (oven method)*.—Use A.S.T.M. Method D 460-46, section 10 (1952, Part 7, p. 584). Calculate the result as the percentage of moisture or percentage of solids in the sample.

(26) *Naphthenic acids*.—Weigh accurately 3 to 5 g. of sample (or more if naphthenic acid content is less than 40 percent) into a 250-ml. conical flask. Add 25 ml. of alcohol (80 percent by volume) and reflux the mixture for 1 hour. Decant and save the extract and repeat the refluxing and decantation procedures with alcohol until the last extract does not leave a residue upon evaporation. Combine the extracts and evaporate to dryness on a steam bath. The residue consists of sodium naphthenates and other alcohol-soluble matter.

Dissolve the residue in water, transfer to a separatory funnel with the aid of wash water. Add an excess of copper sulfate solution (10 percent) to precipitate all the naphthenates as copper salts. Extract the material with benzene until a colorless extract is obtained. To ensure that sufficient copper is present, add more copper sulfate solution to the aqueous solution and again extract with benzene. If this extract is colored, add it to the combined benzene extracts. Wash the combined extracts with water, shaking lightly. Evaporate to dryness in a tared dish, dry for 1 hour at 105°C., cool, and weigh as copper naphthenate.

Weigh a portion of the dried copper naphthenate into a tared porcelain crucible, ignite gently with a bunsen flame, cool, and weigh the residue as copper oxide. Calculate the copper content of the copper naphthenate (copper = 0.798 of copper oxide) in the ignited portion and use this factor to calculate the percentage of naphthenic acids in the sample (total weight of copper naphthenate less the weight of copper present divided by the weight of the sample).

Note: This method is based on that presented in the Institute of Petroleum standard methods (*28*). The term naphthenic acids as referred to here means the carboxylic acids occurring in crude petroleum which combine with copper to give benzene soluble salts.

(27) *Nitrogen (modified Gunning method)*.—Place 0.7 to 3.5 g. (according to nitrogen content) of dried sample in a Kjeldahl digestion flask. Add 30 ml. of concentrated sulfuric acid containing 1 g. of dissolved salicylic acid; shake until thoroughly mixed; and allow to stand at least 30 minutes or until complete solution results (shake frequently). Add 5 g. of sodium thiosulfate and heat gently for 5 minutes. Cool, add 10 g. of potassium sulfate or anhydrous sodium sulfate, and heat gently until foaming ceases. Slowly increase the heat until the acid boils and digest until the oxidation is complete, continuing for a time after the mixture is colorless or nearly so.

After cooling, dilute with approximately 200 ml. of water and add a few pieces of granulated zinc to prevent bumping, add sufficient sodium hydroxide solution (450 g. per liter) to make the reaction mixture strongly alkaline (50 to 100 ml. is usually sufficient), pouring it down the side of the flask so that it does not mix all at once. Connect the flask to the condenser by means of a Kjeldahl connecting bulb, taking care that the tip of the condenser extends below the surface of standard acid in the receiver. Mix the contents by shaking and distill until all ammonia has passed over into a measured quantity of the standard acid (amount of acid must be adjusted according to expected nitrogen content). The first 150 ml. of distillate usually contains all the ammonia. Titrate the excess of standard acid with standard alkali (0.1N NaOH) using methyl red as the indicator, and from the amount of acid consumed calculate the nitrogen content on the basis of the total sample.

Note: This method is essentially the same as a previously published procedure (ref. *27*, method 2.26, p. 13).

(28) *Proteins*.—Multiply the percentage of total nitrogen as determined by test 27 by the factor 6.3 to obtain the percentage of proteins in the sample.

Note: This factor varies for different type proteins (from 5.6 to 6.7). However, 6.3 is considered to be of sufficient accuracy for the purposes of this estimation.

(29) *Rosin (McNicoll method)*.—Follow the procedure in A.S.T.M. Method D 460-46, sections 29-32 (1952, Part 7, p. 589). Calculate the percentage of rosin on the basis of the original sample.

(30) *Sulfuric anhydride (organically combined)*.—Determine as directed in A.S.T.M. Method D 500-45, sections 23-26 (1952, Part 7, p. 632).

(31) *Synthetic detergent (colorimetric method)*.—Weigh accurately 1 g. of the sample. Dissolve and transfer to a 100-ml. volumetric flask. Fill to the mark. Measure an aliquot containing from 0.1 to 0.4 mg. of synthetic detergent into a separatory funnel and dilute to approximately 20 ml. with water. Neutralize the solution with dilute hydrochloric acid, using 3 to 4 drops in excess. Add 1 ml. of methylene blue solution (0.1-percent aqueous solution of methylene blue hydrochloride) and mix thoroughly. Add 20 ml. of chloroform, shake gently for 1 minute, let stand 5 minutes, and draw off the chloroform layer into a second separatory funnel. Add 20 ml. of distilled water to the second funnel and shake gently for 1 minute, let stand 5 minutes, and filter through cotton into a 100-ml. volumetric flask. Make three additional extractions with chloroform and subsequent washings as before. Wash the filter funnel and cotton with chloroform, fill the flask to mark with chloroform, mix, and analyze colorimetrically with a source of light at 652 millimicrons. The instrument should be standardized by using various sized aliquots of a solution of a reference sample of the synthetic detergent being determined, and utilizing the same procedure as described for the sample. Calculate the result as the percentage of synthetic detergent in the sample.

Note: This method is particularly applicable when the amount of synthetic detergent in the sample is small. Quantitative results are dependent upon the availability of a standard or reference sample for calibration purposes. The procedure has previously been published (*29*).

(32) *Synthetic detergent (difference method)*.—Calculate the percentage of synthetic detergent by subtracting the total of the percentages of moisture, alcohol-insoluble matter, fatty matter in synthetic detergent, and the chlorides (as sodium chloride) in the alcohol-soluble matter, from 100.

(33) *Synthetic detergent (volumetric method)*.—The method given here is adapted from a published procedure (*30*). It is based upon the principle that alkyl aryl sulfonates will react with p-toluidine hydrochloride in aqueous solution to form a complex amine salt which is soluble in carbon tetrachloride. Upon titration with standard alkali, the complex splits, yielding the organic sulfonic acid which reacts with the standard alkali. The results are expressed in terms of a factor, unless the molecular weight of the alkyl aryl sulfonate is known or a reference sample is available in which cases the percentage of synthetic detergent may be calculated directly. The determination is conducted as follows:

Weigh accurately 3 to 4 g. of solid sample (or sufficient liquid sample to yield this amount of solids) into a 250-ml. separatory funnel. Add 50 ml. of carbon tetrachloride and 100 ml. of an aqueous solution containing 3.40 g. of paratoluidine hydrochloride. Stopper the funnel and shake for 5 to 10 minutes or until all the solid matter has disappeared. Let settle and draw off the carbon tetrachloride layer into a 500-ml. glass-stoppered Erlenmeyer flask. Repeat the extraction with 25 ml. of carbon tetrachloride.

Combine the extracts and add 100 ml. of 95-percent ethyl alcohol (just previously neutralized with 0.10N sodium hydroxide to a faint purple with m-cresol purple indicator solution). Titrate with standard 0.10N sodium hydroxide using m-cresol purple indicator (0.5 percent) until the emulsion formed by vigorous and intermittent shaking remains lavender. Express the results in terms of milliliters of 0.10N sodium hydroxide required to neutralize 1 g. of original sample.

Best results are obtained where the alkyl aryl sulfonate content of the solids is between 30 and 60 percent. The method is applicable to any compound containing an alkyl group combined with such aryl groups as benzene, toluene, phenol, or naphthalene.

(34) *Triethanolamine*.—The following procedure was devised by the authors and gave satisfactory results when other inorganic salts (especially sodium sulfate) were not present:

Weigh accurately a sample containing 1 to 2 g. solids into a 250-ml. beaker and dilute with water to 100 ml. Add sufficient concentrated hydrochloric acid to insure a moderate excess. Stir well and let settle (see note below). Decant the aqueous layer into a second beaker, taking care to retain all the undissolved material in the original beaker. Repeat the extraction with several 50-ml. portions of moderately acidified water.

Evaporate the combined extracts to dryness, add 100 ml. of 95-percent alcohol, stir well, and filter thru a tared fritted glass crucible of medium porosity. Wash the beaker and crucible with several portions of alcohol. Dry the crucible for one-half hour at 105°C., cool, and weigh as triethanolamine hydrochloride ($C_6H_{15}O_3N.HCl$). Calculate the percentage of triethanolamine on the basis of the original sample (triethanolamine hydrochloride × 0.8033 = triethanolamine).

Note: Where fatty matter separates and rises to the surface, it is advisable to carry out the extraction in a separatory funnel and draw off the aqueous layer.

(35) *Unsaponifiable matter*.—Use A.S.T.M. Method D 460-46, sections 25-27 (1952, Part 7, p. 589).

(36) *Unsaponified plus unsaponifiable matter*.—Use procedure shown in A.S.T.M. Method D 460-46, sections 22-24 (1952, Part 7, p. 588).

(37) *Unsaponified matter*.—Use A.S.T.M. Method D 460-46, section 28 (1952, Part 7, p. 589).

Appendix III.—References

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Admixtures Approved by the Bureau of Public Roads for Use on Direct Government Contracts in the National Parks and National Forests

Trade Name	*Manufactured by*
Aerolith	L. Sonneborn Sons, Inc., New York, N. Y.
Airalon AEA	Dewey & Almy Chemical Co., Cambridge, Mass.
Ayr-Trap #1	A. C. Horn Co., Long Island City, N. Y.
Ayr-Trap #2	A. C. Horn Co., Long Island City, N. Y.
Darex AEA	Dewey & Almy Chemical Co., Cambridge, Mass.
Durair	Techkote Co., Inc., Inglewood, Calif.
Kutwell 40	Esso Standard Oil Co., New York, N. Y.
Nacconol NR	Allied Chemical & Dye Corp., New York, N. Y.
Nacconol Z	Allied Chemical & Dye Corp., New York, N. Y.
N-Tair	Newport Industries, Inc., New York, N. Y.
Portite	Hopper Products, Inc., New York, N. Y.
Pozzolith 4A	Master Builders Co., Cleveland, Ohio.
Pozzolith 5A	Master Builders Co., Cleveland, Ohio.
Pozzolith 8	Master Builders Co., Cleveland, Ohio.
Pozzolith 10A	Master Builders Co., Cleveland, Ohio.
Protex AEA	Autolene Lubricants Co., Denver, Colo.
Ricco	Richards Sales Corp., Jersey City, N. J.
Santomerse No. 1	Monsanto Chemical Co., St. Louis, Mo.
Santomerse S	Monsanto Chemical Co., St. Louis, Mo.
Sika AER	Sika Chemical Corp., Passaic, N. J.
Spray-O-Bond	Spray-O-Bond Co., Milwaukee, Wis.
Specs	E. L. Moore Co., Pasadena, Calif.
Ultrawet 30E	The Atlantic Refining Co., Philadelphia, Pa.
Ultrawet K	The Atlantic Refining Co., Philadelphia, Pa.
Veriset	Verapon, Inc., New York, N. Y.
Vinsol NVX	Hercules Powder Co., Wilmington, Del.

The admixtures included above are approved subject to the following provision: Either prior to or at any time during construction the engineer may require that the admixture selected by the contractor be further tested to determine its effect upon the strength of the concrete. When so tested, compressive strength at 7 days of concrete made with the cement and aggregates and in the proportions to be used in the work and containing the admixture under test in an amount sufficient to produce from 3 to 6 percent entrained air in the plastic concrete, shall not be less than 88 percent of the strength of concrete made with the same materials and with the same cement content and consistency but without the admixture.

The percentage reduction in strength shall be calculated from the average strength of at least five standard 6- by 12-inch cylinders of each type of concrete. Specimens shall be made and cured in the laboratory in accordance with A.S.T.M. Standard Method of Making and Curing Concrete Compression and Flexure Specimens in the Laboratory, C 192-49, and shall be tested in accordance with A.S.T.M. Standard Method of Test for Compressive Strength of Molded Concrete Cylinders, C 39-49. The percentage of entrained air shall be determined in accordance with A.S.T.M. Tentative Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method, C 231-49 T.

The contractor will be required to follow an approved procedure for adding the specified amount of air-entraining admixture to each batch and will be held responsible for its uniform operation during the progress of the work. The contractor shall provide separate approved scales for such admixtures as are to be proportioned by weight and accurate measures for such admixtures as are to be proportioned by volume.

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Act III.—Uniform Motor-Vehicle Civil Liability Act (1944). 10 cents.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act (revised 1952). 15 cents.

Act V.—Uniform Act Regulating Traffic on Highways (revised 1952). 20 cents.

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United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

MISCELLANEOUS PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.

Construction of Private Driveways, No. 272MP (1937). 10 cents.

Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.

Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.

Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.

Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.

Highway Accidents (1938). 10 cents.

Highway Bond Calculations (1936). 10 cents.

Highway Bridge Location, No. 1486D (1927). 15 cents.

Highway Capacity Manual (1950). 65 cents.

Highway Needs of the National Defense, House Document No. 249 (1948). 75 cents.

Highway Practice in the United States of America (1949). 75 cents.

Highway Statistics (annual):
1945, 35 cents.
1946, 50 cents.
1947, 45 cents.
1948, 65 cents.
1949, 55 cents.
1951, 60 cents.
1952, 75 cents.

Highway Statistics, Summary to 1945. 40 cents.

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Manual on Uniform Traffic Control Devices for Streets and Highways (1948). 75 cents.

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.

Public Control of Highway Access and Roadside Development (1947). 35 cents.

Public Land Acquisition for Highway Purposes (1943). 10 cents.

Results of Physical Tests of Road-Building Aggregate (1953). $1.00.

Roadside Improvement, No. 191MP (1934). 10 cents.

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Standard Plans for Highway Bridge Superstructures (1953). $1.00.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1953

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$5,032	$9,557	$4,501	138.3	$7,369	$3,684	95.2	$41,793	$21,297	438.7	$58,719	$29,482	672.2
Arizona	1,283	1,894	1,267	57.4	52	47		6,748	4,762	85.9	8,694	6,076	143.3
Arkansas	2,352	9,179	5,045	325.1	5,524	2,754	123.1	9,257	4,664	219.5	23,960	12,463	667.7
California	1,452	4,965	2,827	59.8	3,907	2,169	12.9	92,358	44,474	217.2	101,230	49,470	289.9
Colorado	1,923	3,545	1,986	70.7	1,215	683	35.4	18,314	9,932	181,5	23,074	12,601	287.6
Connecticut	6,760	450	275	2.0	1,672	883	1.7	8,581	4,228	25.9	10,703	5,386	29.6
Delaware	791	4,877	2,462	28.7	355	177	6.3	3,599	1,792	18.8	8,831	4,431	53.8
Florida	4,327	11,454	5,797	157.9	7,308	4,000	136.6	18,766	9,657	262.7	37,528	19,454	557.2
Georgia	3,979	15,116	7,740	335.6	7,382	3,688	109.4	31,897	14,936	446.7	54,395	26,364	891.7
Idaho	2,117	5,757	3,595	92.2	3,386	2,127	41.5	9,067	5,665	158.6	18,210	11,387	292.3
Illinois	10,264	21,494	11,052	153.7	13,321	6,600	53.4	54,918	29,222	286.2	89,733	46,874	493.3
Indiana	11,656	28,168	15,432	133.4	9,205	4,617	45.4	16,710	8,870	53.3	54,083	28,919	232.1
Iowa	2,119	10,057	5,439	248.0	5,683	2,936	145.1	13,303	7,749	429.7	29,043	16,124	822.8
Kansas	2,777	9,905	4,956	790.4	2,756	1,392	293.2	13,423	6,554	657.6	26,084	12,902	1,741.2
Kentucky	1,779	9,609	5,148	108.7	5,483	2,741	79.4	20,918	10,974	203.5	36,010	18,863	391.6
Louisiana	2,164	15,862	7,930	122.0	4,621	2,340	37.2	26,997	13,064	141.9	47,480	23,334	301.1
Maine	1,050	4,077	2,176	27.1	760	380	4.7	11,265	5,644	84.7	16,102	8,200	116.5
Maryland	8,177	2,617	1,412	34.1	4,663	2,230	24.8	8,668	4,803	32.2	15,948	8,445	91.1
Massachusetts	4,951	1,543	906	6.4	7,159	3,562	10.3	40,415	18,710	23.9	49,117	23,178	40.6
Michigan	1,982	24,592	12,556	327.2	8,786	4,616	131.7	48,868	20,802	163.3	82,246	37,974	622.2
Minnesota	3,693	9,398	5,098	765.1	2,008	1,031	41.5	12,705	6,712	201.8	24,111	12,841	1,008.4
Mississippi	946	10,949	5,589	299.7	3,672	1,804	132.7	21,595	10,932	520.1	36,216	18,325	952.5
Missouri	5,568	13,435	6,814	763.7	4,867	2,510	153.1	46,714	22,921	336.4	65,016	32,245	1,253.2
Montana	3,819	11,362	6,958	258.9	4,803	2,893	96.1	15,728	9,525	248.4	31,893	19,376	603.4
Nebraska	6,228	22,629	11,763	682.6	4,306	2,371	104.8	8,562	4,872	227.5	35,497	19,006	1,014.9
Nevada	2,504	5,485	4,590	80.6	457	383	15.1	4,807	3,987	101.7	10,749	8,960	197.4
New Hampshire	2,727	1,780	890	10.1	1,084	537	3.6	4,272	2,254	25.2	7,136	3,681	38.9
New Jersey	2,779	5,769	2,859	56.2	1,834	877	5.1	30,651	14,654	24.9	38,254	18,390	86.2
New Mexico	566	1,284	794	38.2	2,163	1,374	45.9	9,443	5,911	204.8	12,890	8,079	288.9
New York	13,584	54,812	29,141	100.8	48,079	23,530	19.5	141,827	66,016	386.5	244,718	118,687	506.8
North Carolina	3,048	19,078	9,343	330.2	3,461	1,609	73.6	29,979	14,181	352.9	52,518	25,133	756.7
North Dakota	2,988	3,019	1,517	504.8	788	394	62.6	6,437	3,371	347.5	10,244	5,282	914.9
Ohio	2,605	21,678	7,888	125.2	12,758	6,506	37.4	75,916	36,198	88.9	110,352	50,592	251.5
Oklahoma	4,402	14,136	7,771	195.9	6,839	3,715	87.7	13,995	7,436	187.7	34,970	18,922	471.3
Oregon	813	452	266	6.5	3,461	1,957	55.9	9,189	5,568	134.8	13,102	7,791	197.2
Pennsylvania	3,591	13,425	5,479	4.1	24,965	12,183	44.0	79,384	38,923	156.1	117,774	56,585	204.2
Rhode Island	1,805	2,569	1,284	33.8	307	153		9,797	4,896	24.6	12,673	6,333	58.4
South Carolina	1,882	10,537	5,879	201.5	2,005	977	105.8	13,435	6,673	262.4	25,977	13,529	569.7
South Dakota	957	5,178	3,060	305.2	1,760	988	70.7	6,606	3,689	377.2	13,544	7,737	753.1
Tennessee	2,565	10,249	5,102	297.0	8,047	4,037	192.7	29,074	13,187	225.8	47,370	22,326	715.5
Texas	6,495	2,266	1,183	20.4	12,968	6,592	355.8	52,420	28,525	844.3	67,654	36,300	1,220.5
Utah	464	1,749	1,350	27.0	1,325	993	22.1	8,822	6,769	121.9	11,896	9,112	171.0
Vermont	536	3,216	1,727	33.5	296	148	5.5	6,048	3,067	27.4	9,560	4,942	66.4
Virginia	701	8,594	3,787	109.9	5,985	2,663	88.0	26,853	12,966	165.2	41,432	19,416	363.1
Washington	851	7,748	4,094	128.6	1,480	823	16.6	17,325	9,128	112.2	26,553	14,045	257.4
West Virginia	3,360	7,445	3,774	59.1	4,877	2,457	3.5	10,683	5,351	46.7	23,005	11,582	109.3
Wisconsin	2,075	9,402	5,088	130.2	3,521	1,805	91.4	20,907	10,562	169.8	33,830	17,455	391.4
Wyoming	1,429	605	390	12.6	1,031	666	22.8	7,193	4,506	127.2	8,829	5,562	162.6
Hawaii	1,173	3,054	1,495	4.8	343	169	3.0	9,818	4,686	12.3	13,215	6,350	20.1
District of Columbia	845	7,488	3,504	5.5	115	58	.8	12,538	5,779	2.2	20,141	9,341	8.5
Puerto Rico	3,487	10,561	4,865	51.9	617	234	2.1	12,939	6,187	40.1	24,117	11,286	94.1
TOTAL	164,421	494,070	255,844	8,862.3	270,829	138,063	3,346.7	1,251,527	627,231	10,236.3	2,016,426	1,021,138	22,445.3

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